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PROJECT ADMINISTRATION DATA SHEET

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Title: Evaluation of the Material Properties of Stitch Bonded Geotextiles

ADMINISTRATIVE DATA OCA Contact E. Faith Gleason X-4820

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SPONSORED PROJECT TERMINATION/CLOSEOUT SHEET

Date 6/9/88

Project No. E-20-666 School/~~Lab~~ CE

Includes Subproject No.(s) N/A

Project Director(s) ~~H.~~ N. D. Williams GTRC/~~GIT~~

Sponsor Exxon Chemical Co.

Title Evaluation of the Material Properties of Stitch Bonded geotextiles

Effective Completion Date: 10/11/87 (Performance) 10/11/87 (Reports)

Grant/Contract Closeout Actions Remaining:

- [ ] None
- [x] Final Invoice or Copy of Last Invoice Serving as Final
- [ ] Release and Assignment
- [x] Final Report of Inventions and/or Subcontract:
  Patent and Subcontract Questionnaire sent to Project Director [x]
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FINAL REPORT

ENGINEERING PROPERTIES OF STITCH BONDED GEOTEXTILES

Prepared by
Kenneth L. Still
Neil D. Williams, Ph.D., P.E.
The Georgia Institute of Technology
School of Civil Engineering

Prepared For
Exxon Chemicals
Geotextile Fabrics Division

November 1987

GEORGIA INSTITUTE OF TECHNOLOGY
A UNIT OF THE UNIVERSITY SYSTEM OF GEORGIA
SCHOOL OF CIVIL ENGINEERING
ATLANTA, GEORGIA 30332

1987

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CHAPTER 1

INTRODUCTION

Scope and Objectives

Geosynthetics have been widely used to provide tensile reinforcement of soil structural elements. These geosynthetics may divided into two primary types, geotextiles and geogrids. Geotextiles are woven or nonwoven fabrics composed of synthetic polymers such polypropylene or polyester, which are used in civil engineering works. Stitchbonded geotextiles consist of multiple light weight woven or nonwoven geotextiles stitched together to form a multiple layer fabric with the design strength, modulus, and durability properties.

The primary focus of the research project discussed herein was to evaluate the interaction between stitchbonded geotextiles and soil. A comprehensive laboratory study was performed to evaluate the index and performance properties of stitchbonded geotextiles. These analyses provided the basis for an evaluation of current design methodologies for reinforced steepened slopes using geosynthetic reinforcement.

Terminology

The use of geosynthetics in many types of geotechnical structures has become commonplace. Applications such as those shown in Figure 1-1 are readily suited to soil reinforcement using geosynthetics. The use of polymeric materials as a civil engineering building material has established the need for terminology that is product type, testing and application specific. Key terms that are essential to understanding the performance and testing of geosynthetics are defined and discussed below.

The term geosynthetic refers to a wide range of polymeric materials that when used together with conventional geotechnical building materials such as soil and rock form an improved composite system. Geosynthetics are frequently further divided into four subgroups; geogrids, geotextiles, geomembranes and geocomposites.

Geogrids refer to continuous extruded polymeric sheets that are expanded into grid, net and web structures. The main use of geogrids is for soil reinforcment in applications such as improved embankments and retaining walls. The primary mechanism involved in geogrid reinforcement is that of interlock between the soil particles and grid structure.

The term geotextile is used to define a group of geosynthetics that are commonly used for separation, filtration and retention of soils. Woven geotextiles are typically comprised of extruded slit-film or continuous filament polymers that are woven together much like clothing textiles. Their primary use is the separation of soils as in the case of embankments over soft subgrades. The composite system gathers its strength from the geotextile being confined by fill overburden and placed in tension, thus allowing the product to "bridge" over softer underlying material. In railroad applications, woven and nonwoven geotextiles are used to prevent costly ballast material from being lost as it "punches" into softer subgrade material as well as to provide a drainage path for water arising from the build-up of excess pore pressure caused by rail loading. In this case, the geotextile functions mainly as a separation barrier, but also provides tension reinforcement and drainage as mentioned above. Nonwoven geotextiles are manufactured from continuous filament or staple materials. onto a moving conveyor belt. The fine polymer threads fall in a random manner, but with a relatively uniform thickness. The extruded threads are then bonded together by any one of several methods such as needle-punching, heat calendering, heat burnishing and chemical bonding.

The American Society of Testing and Materials (ASTM) defines a geomembrane as: "Any impermeable membrane used with foundation, soil, rock, earth or any other geotechnical engineering-related material, as an integral part of a man-made project, structure or system." However, the author feels this definition should be reworded as, "Any **relatively** impermeable **polymeric** membrane .....," since all products of this type on the market today have a measurable value of hydraulic conductivity and are therfore not impermeable.

Geomembranes are most commonly used in containment applications. By-products from industrial processes, mine tailings, various municipal sludges are typically placed in landfills or surface impoundments lined with geomembranes.

The use of geocomposites has become quite popular in recent years. The term geocomposite is used to describe a multi-layer system consisting, generally, of a grid or net structure used as a drainage corridor covered on one or both sides by a nonwoven geotextile to serve as a filtration barrier to reduce clogging of the drainage net. Some more unique composites incorporate a polymeric cuspaded core structure often resembling an egg carton wrapped in a suitable filter fabric. This type of composite is commonly used to replace granular filter material behind retaining walls and other structures where drainage is needed.

In general, the performance of geosynthetics depends on the mechanical, chemical and hydraulic properties of the product. To date, three types of testing are used in describing a geosynthetic's quality and suitability for a given application. These three type of tests are: control tests, index tests and design or performance tests.

Control tests are performed by the manufacturer in-house as a measure of quality control. Since the polymer materials are batched, tests are performed to maintain mechanical properties such as density, thickness and specific gravity within acceptable limits. These tests are relatively simple and in some cases can be performed on-line. Unfortunately, to date, no standards exist for control tests. Thus, comparison of the same type of product between two manufacturers can often be misleading.

Index tests are performed under controlled laboratory conditions using (when available) standard procedures set forth by ASTM. The purpose of index tests is to allow the comparison of similar products produced by different manufacturers. However, historically, when index tests are performed in-house, results are often published in design literature based on values other than minimum roll values which can be misleading to the geotechnical engineer. Unconfined tensile strength, trapezoidal shear, tension creep, mullen burst and puncture strength are typical index tests. However, index tests are performed under a prescribed set of boundary conditions which do not typically model the field conditions. Since the results of the analyses are dependent on the boundary conditions, the index test results should not be used for design.

Performance tests are carried out with equipment and enviroments that more closely model in-situ conditions with respect to soil type, confinement and mechanism of load transfer. Typical performance tests include: interface friction analyses, direct shear, triaxial shear, transmissivity, hydraulic conductivity ratio and confined tensile strength (pullout). Performance tests should be used for design whenever possible. However, the cost of such a testing program is often beyond the budget and scope of small projects. In such cases, the results of index test data may be the only information available to the engineer and should only be used with extreme caution and good judgment.

Limiting equilibrium analysis procedures are typically used in the design of reinforced earth slopes. These design methods assume a failure mechanism and failure surface. With respect to geosynthetic reinforced structures, three modes of failure are normally considered: creep, pullout and rupture.

Polymeric materials and soils exhibit time dependent deformation. The time dependent deformation for a specimen under constant stress is termed creep. In comparison to soils, geosynthetics are highly susceptible to creep. Fortunately, the confinement of the geosynthetic by soil reduces creep effects. Confined tensile creep is the creep of a material constrained by a frictional medium (21).

If a geosynthetic-soil specimen is loaded at an increasing rate, the specimen will fail by either pullout or rupture. If the frictional forces and passive resistance are insufficient to constrain the geosynthetic, and the forces developed within the geosynthetic are less than those required for failure, the geosynthetic will slide along the soil interface. This mode of failure is refered to as pullout. This sliding may also occur along a weak plane in the soil matrix, but near the geosynthetic. In this case, the geometry of the geosynthetic is such that it interlocks sufficiently with the soil to cause the sliding failure to move into the soil matrix. However, if the confinement offered by the soil matrix is sufficient to restrain the geosynthetic from sliding at the interface or at some distance, the failure is termed rupture in which case the tensile strength of the geosynthetic is exceeded (21).

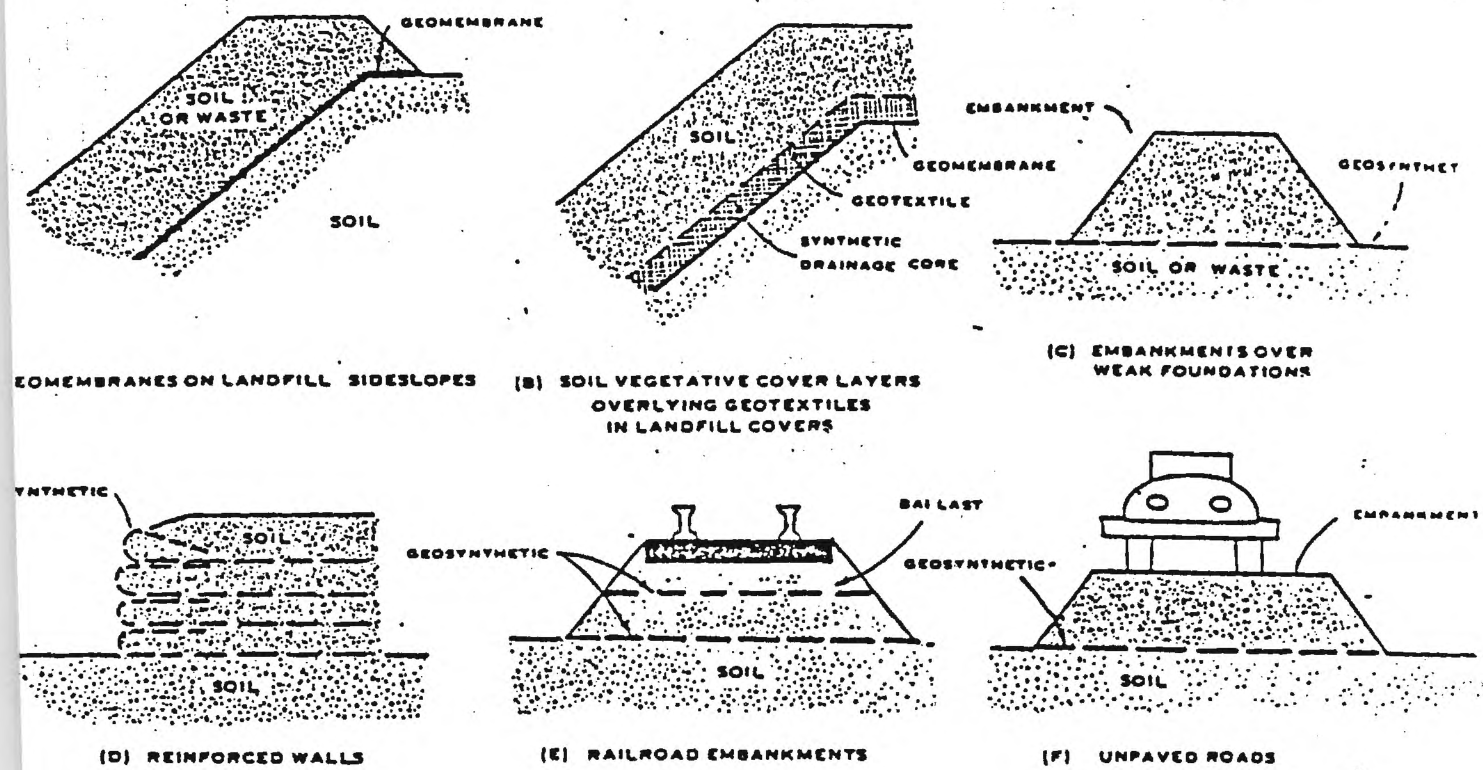


FIGURE 1. COMMON EXAMPLES OF THE APPLICATION OF INTERFACE FRICTION PROPERTIES FOR THE DESIGN OF GEOSYNTHETICS IN ENGINEERING WORKS.

Figure 1- 1 Common Geosynthetic Reinforcement Applications (after Williams and Houlihan, 1987).

CHAPTER 2

BACKGROUND

The products used in this study were provided by the Exxon Chemical Company under the guidance of Mr. John N. Paulson. The products are refered to as stitch-bonded composites and are currently in the development stage. The products tested are named GTF 450, 500 and 800. GTF stands for GeoTextile Fabric. GTF 450 is a single layer product, GTF 500 is a 3 layer product and GTF 800 is composed of 4 layers.

Stitch-bonding is a method of sewing multiple geotextile layers together using knitting manufacturing techniques. The sewn stitches consists of a series of chainstitches installed in closely spaced rows. This process serves to mechanically combine the individual layers into one composite. Variations in both the stitch line spacing and the number of stitches per inch of stitchline are possible (23).

For this study, combinations of a medium strength slit-film polypropylene geotextile stitch-bonded to form three and four layer woven composites were tested. In addition, a woven/nonwoven composite product consisting of a slit film polypropylene woven layer sandwiched between two polypropylene needlepunched nonwoven layers was tested. All composites tested were sewn together with 500 denier polyester sewing thread with 2.3 - 3.0 stitches per centimeter, and one centimeter between stitchlines. Tables (2-1),(2-2), and (2-3) list the maunufacturer's index test results for the three products. Figures (2-1) through (2-5) are comparative plots showing the effect of multiple layers.

As the stitching yarn size increases, the strain required to straighten the yarn and provide tensile strength also increases. This additional strain results in "off-set strain" with respect to a wide width stress-strain curve.

Evaluation of off-set strain is important in soil-geotextile interaction, as evidenced by the total resulting strain required to achieve a desired stress level in the material. Conversely, the maximum stress level that can be achieved in that same product may be limited by the allowable strain in the structure. The term "crimp" is used the textile industry to describe the strain required to straighten out a yarn, allowing it to take load. All woven geotextiles exhibit some degree of off-set strain. Figure (2-3) presents the graphical definition of off-set strain in a typical geotextile stress-strain curve (23).

In addition to the added strength of a composite, increased flexural rigidity is of great benefit. Construction time of projects over extremely soft subgrades can be greatly reduced by using geotextiles with high rigidity. The stiff, bordy nature of the stitch-bonded products allows construction crews to walk on the product with minimum delay.

Another advantage of stitch-bonded composites, is the ability to combine hydraulic properties of a nonwoven with the reinforcement strength properties of a woven geotextile into one product. Build up of excess pore pressure within reinforced slopes can be significantly reduced by providing multiple drainage paths, thereby increasing stability. Figure (2-6) presents a typical slope crossection utilizing stitch-bonded composites for reinforcement and drainage.

Table 2-1

Woven Geotextile Physical Properties

	Physical Properties	Grab Tensile Strength ASTM 1682	Wide Width Strip Tensile ASTM D4595	Secant Modulus at 10% Strain ASTM D4595	Trapezoid Tear ASTM D1117	Puncture ASTM D751	Burst ASTM D751
Geotextile	Single Layer	890 N (200 lb)	30 KN/m (170 lb/in)	117 KN/m (670 lb/in)	400 N (90 lb)	445 N (100 lb)	3272 KPa (475 psi)
	3 Layer Composite	3335 N (705 lb)	77 KN/m (440 lb/in)	613 KN/m (3500 lb/in)	890 N (200 lb)	1780 N (400 lb)	10,000 KPa (1500 psi)
	4 Layer Composite	4000 N (900 lb)	104 KN/m (590 lb)	875 KN/m (5000 lb/in	980 N (220 lb)	2000 N (450 lb)	10,000 KPa (1500 psi)

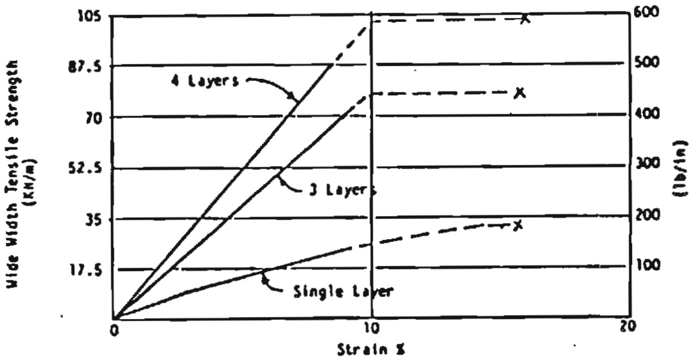
Table 2-2 Geotextile Flexural Rigidity

	Weight W	Bending Length (c)	Flexural Rigidity $G = Wxc^3$
Lightweight Single layer Slit tape woven	$15.9 mg/cm^2$ (4.7 oz/sy)	7.2 cm	6185 mg-cm
High Strength Multi-Filament woven	$72.5 mg/cm^2$ (21 oz/sy)	5.7 cm	13426 mg-cm
4 layer Stitch-bonded woven composite	$67 mg/cm^2$ (19.8 oz/sy)	16.3 cm	292560 mg-cm

Table 2-3

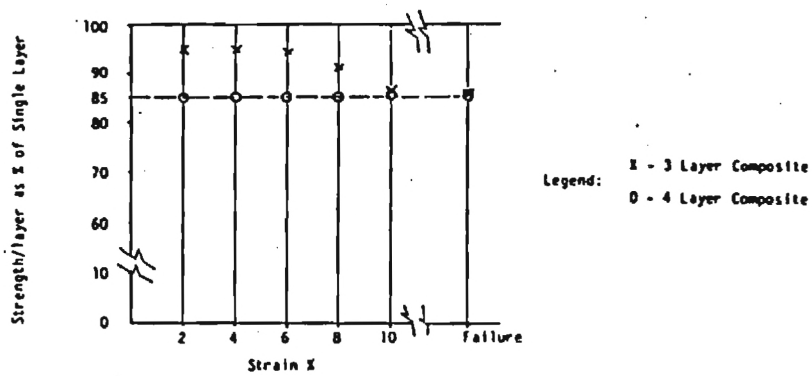
Woven/Non-Woven Geotextile Physical Properties

	Physical Properties	Grab Tensile Strength ASTM 1682	Wide Width Strip Tensile ASTM D4595	Secant Modulus at 10% Strain ASTM D4595	Trapezoid Tear ASTM D1117	Puncture ASTM D751	Burst ASTM D751
Geotextile	Woven Layer	890 N (200 lb)	30 KN/m (170 lb/in)	117 KN/m (670 lb/in)	400 N (90 lb)	445 N (100 lb)	3272 KPa (475 psi)
	Non-Woven Layer	711 N (160 lb)	7.88 KN/m (45 lb/in)	1.75 KN/m (10 lb/in)	310 N (70 lb)	290 N (65 lb)	1550 KPa (225 psi)



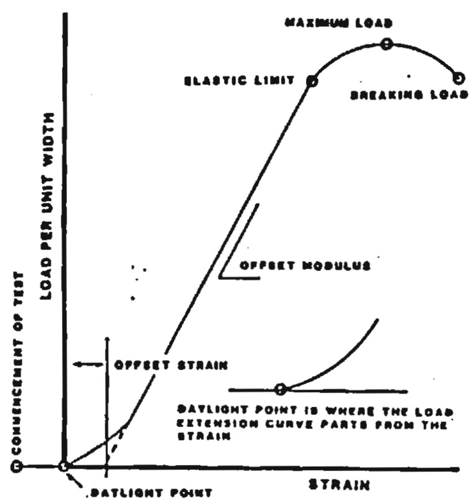
Wide width tensile strength comparisons

Figure 2-1



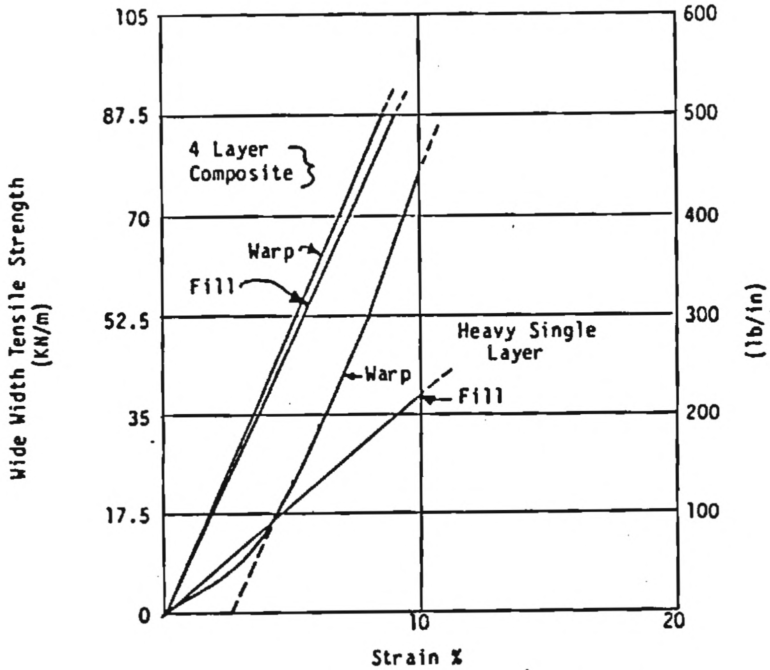
Stitchbonding efficiency as measured by individual layer strength retention using wide width testing

Figure 2-2



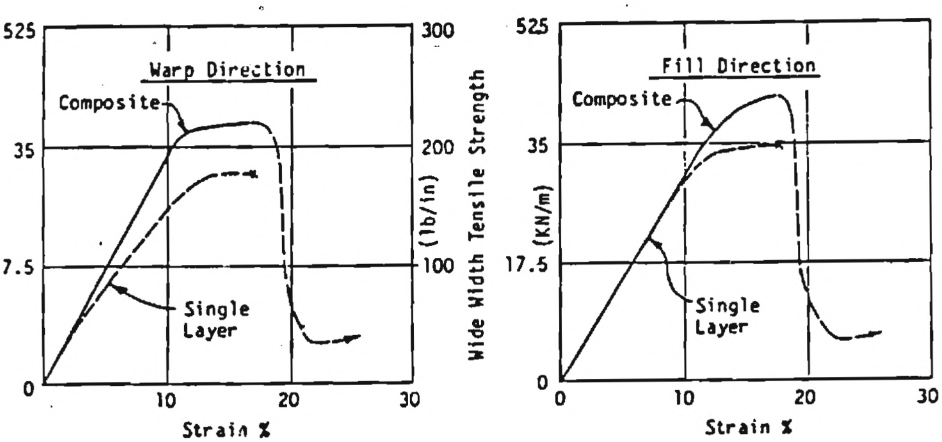
LOAD - STRAIN CURVES FOR GEOTEXTILES FROM MYLES

Figure 2-3



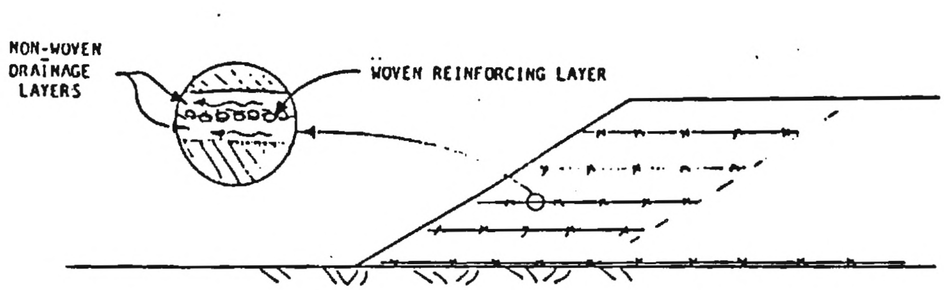
Comparison of single layer and multiple layer composite load/strain characteristics

Figure 2-4



Comparative load/strain curves for woven/non-woven composite and woven geotextile

Figure 2-5



Embankment reinforcing and drainage combination geotextile

Figure 2-6

CHAPTER 3

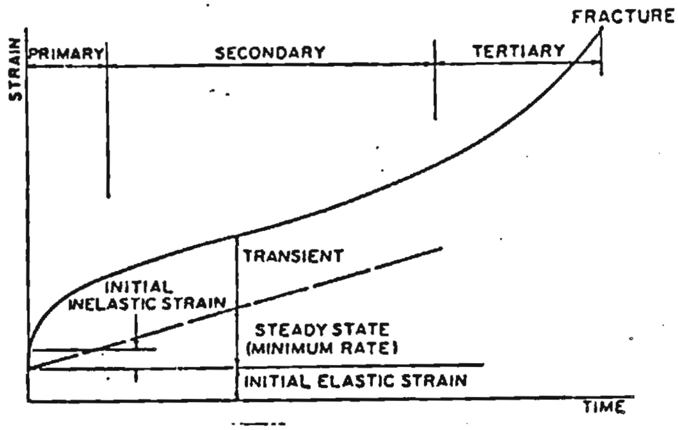
FACTORS AFFECTING PULLOUT AND CREEP OF GEOTEXTILES IN A SOIL MATRIX.

Introduction

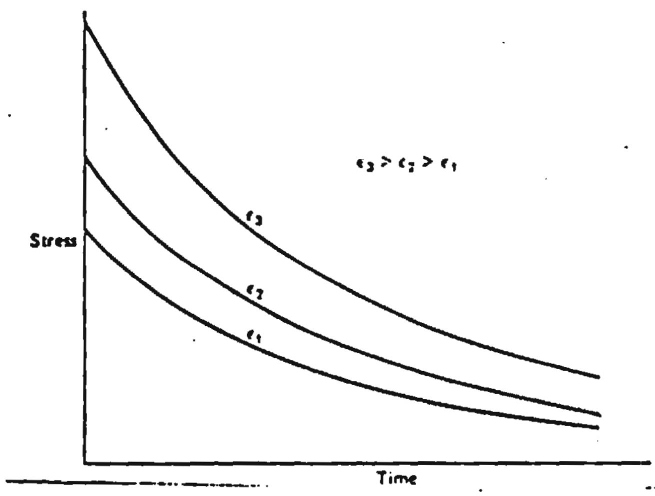
The design of reinforced soil structures requires information specific to the soil, geosynthetic and the soil-geosynthetic system.

Soils and geosynthetics exhibit both creep and stress relaxation. Figure (3-1) presents the difference between creep and stress relaxation for a soil, and a similar relation exists for geosynthetics as seen in Figure (3-2) . Mitchell, 1976, defined creep in soils as the, "time-dependent shear strains and/or volumetric strains that develop at a rate controlled by the viscous resistance of the soil structure." Creep rupture occurs when a soil, geosynthetic or soil-reinforcement system fail under a sustained stress less than the peak stress measured in a sample loaded in a few minutes or hours.

Total creep strain for soils is generally divided into volumetric and deviatoric creep. Volumetric creep, or secondary compression, results from a hydrostatic stress condition under drained conditions (21). Deviatoric strain develops when shear stresses are imposed on a soil. Deformation as a function of time may occur if shear stresses are less than those required for the soil to yield. This strain at constant deviatoric stress is termed deviatoric creep. In the design of geosynthetic reinforced structures, changes in shape or deflection and deformation of the structure are functions of the deviatoric creep of the system (21).



(a) The three stages of creep (after Finnie and Heller, 1959)



(b) Stress relaxation (after Mitchell, 1976)

Figure 3-1 The stages of creep. Stress relaxation.

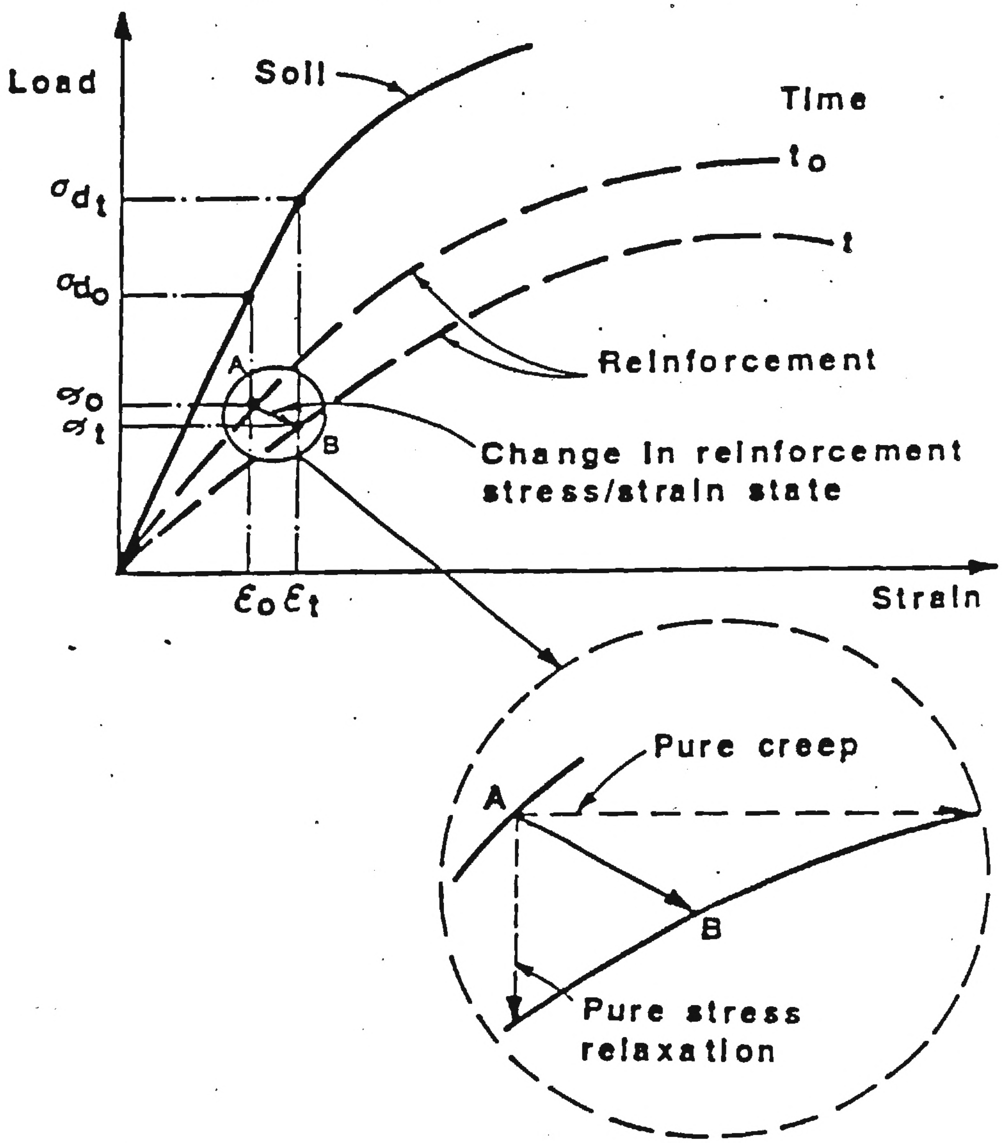


Figure 3-2. Influence of reinforcement creep and stress relaxation on strain developed under working conditions (after Bonaparte et al., 1985).

In the design of geotechnical structures, defining what is to be termed failure of a system is necessary. Limiting equillibrium analyses consider a system at a state of stress just below failure. Traditionally, the Mohr-Coulomb theory is used to describe the failure envelope in soils. The Mohr-Coulomb theory is generally stated as:

$$= c + v^{*}\tan\phi$$

where:		- shear stress at failure	(FL-1)
	c	- cohesion intercept	(FL-1)
		- normal stress	(FL-1),and
	ϕ	- angle of internal friction	(degrees)

The above equation may be expressed in terms of total or effective stress and used as an approximation of a soil's yield function.

Mitchell, 1976, has suggested a more complete relation for defining shearing resistance of a soil in the form:

Shearing resistance = F(e,ϕ,C,s',c',H,T,E,E',S)

where:	e	- void ratio
	ϕ	- angle of internal friction
	C	- composition
	s'	- effective stress
	c'	- effective cohesion
	H	- stress history
	T	- temperature
	E	- strain
	E'	- strain rate
	S	- structure

As Mitchell points out, however, all of the parameters may not be independent and their quantitative functional forms are not all known.

Table (3-1) taken from Newman, 1986, is a summary of key factors which influence pullout and creep of geosynthetics in a soil matrix.

TABLE 3-1

FACTORS INFLUENCING PULLOUT AND CREEP

OF GEOSYNTHETICS IN A SOIL MATRIX

SOIL CHARACTERISTICS

- Soil Type
- Void ratio / Relative density
- Angle of internal friction
- Stress state
- Stress History
- Soil structure & fabric

GEOSYNTHETIC CHARACTERISTICS

- Geosynthetic type
- Geometry and surface roughness
- Interface friction parameters
- Tensile elastic modulus
- Confined tensile strength

EXTERNAL FACTORS

- Temperature
- Pore fluid
- Ultraviolet radiation
- Chemical degradation
- Construction methods
- Time

Soil Characteristics

Soil Type

The type of soil used as backfill for soil reinforced structures is often site and project dependent. In the interest of stability, cohesionless materials are often preferred and specified. However, depending on the nature of the structure itself, cohesive materials may be used.

Cohesionless materials offer a higher angle of internal friction than that of cohesive materials and therefore interlock better with the geosynthetic elements. In addition, cohesionless materials offer more desirable drainage characteristics and are not as susceptible to creep behavior.

Good quality cohesionless fill is generally defined as a material that is well-graded, non-corrosive having a high angle of internal friction. A cohesive material, on the other hand, typically consists of fine grain particles having a low angle of internal friction, thus lowering soil-geosynthetic friction. Because of the fine-grained nature of a cohesive material, soil creep and the possibility of hydrostatic pressure build-up is of great concern.

Void Ratio / Relative Density

The void ratio of a soil is defined as the ratio of the volume of voids to the volume of solids:

$$e = Vv/Vs \qquad (3\text{-}1)$$

The void ratio of a soil is particle size, shape and distribution dependent. Karl Terzaghi, in 1960, suggested that shear force is directly proportional to the area of contact between soil particles (21).

Figure (3-3) taken from Newman's study shows the effect of void ratio on angle of internal friction for a concrete sand. For a given soil, as the void ratio decreases, the contact area between soil particles increases. This results in higher strengths at lower void ratios.

Another useful way to characterize the density of a granular soil is with relative density, Dr, defined as:

$$Dr = \frac{e(max) - e}{e(max) - e(min)} \times 100\ \% \qquad (3\text{-}2)$$

where: e(min) = void ratio in densest state

e(max) = void ratio in loosest state

e = in-place void ratio

The verification and control of void ratio in the confined tensile strength program of this study played a crucial role in maintaining repeatable results.

Angle of Internal Friction

The angle of internal friction, delta or angle of shearing resistance is the angle formed by the slope of the Mohr-Coulomb failure envelope. Figure (3-4) presents a typical failure envelope and its relation to the angle of internal friction.

The friction angle integrates all of the factors affecting the shear strength of a soil (distortion, crushing, shifting, rolling, sliding, and dilation), and those factors which depend on the soil mineral (the particle angularity, roughness, sphericity, gradation and relative density (29)).

The interface friction angle of a soil-geosynthetic system is closely related to the friction angle of the soil. The interface friction angle desribes the limiting condition for a given stress state at the soil/geosynthetic interface. This parameter is very important in the design of reinforced slopes and geosynthetic-lined impoundments in that the maximum constructable slope of these structures is often governed by the interface friction angle of the system.

In solid or hazardous waste landfills (Figure 1-1), the consolidation of the waste induces movement of the waste relative to the geomembrane and/or deformation between the geomembrane and the soil. As the deformation continues, increased shear stress is mobilized at the interface between the soil and geomembrane. Thus, the stability of the slope is largely a function of the interface shear strength (34).

Stress State

The shear strength of a soil is largely a function of the state of stress. The concept of stress path is now commonly used to desribe stress state. Stress paths can be used to show successive states of stress which a test specimen undergoes during loading or unloading (12). Stress paths can be expressed in terms of total and/or effective stresses to include the effects of pore pressure. Since an increase in pore pressure reduces the strength of a soil, free-draining granular material is typically specified for fill materials in reinforced earth structures such as retaining walls.

Stress History

Stress history refers to the type and magnitude of stress a soil has been subjected to in the past. Alterations in composition due to weathering as well as void ratio and structure are all a function stress history. In laboratory testing of soils, stress history must be considered and accounted for to accurately model field conditions.

Structure & Fabric

Soil structure is generally defined to include the combined effects of fabric, mineral composition and interparticle bonding.

Gradation is commonly used to determine particle size distribution. Uniform graded soils are soils consisting of particles nearly all the same size. Well-graded refers to soils with a smooth distribution of particle sizes within a given range. Mineral composition and stress history usually dictates a soil's texture with respect to angularity, shape and suseptibility to weathering.

Interparticle bonding of a natural soil is dependent upon the depositional or weathering enviroment as well as stress history. For soils used in construction, these interparticle bonds are often broken down during placement and compaction of fill materials. It is important that laboratory testing programs account for these changes in structure.

GEOSYNTHETIC CHARACTERISTICS

Geosynthetic Type

As stated in the chapter on terminology, geosynthetics may be divided into four categories: grids, textiles, membranes and composites. The pullout and creep charateristics of these materials are mainly governed by the physical geometry, polymer type and method of production.

The most common polymers used in geosynthetics are: polyester, polypropylene, polyethylene, PVC (polyvinylchloride), nylon and kevlar. Each of these polymer types have their own benefits. Approximately 90 percent of all construction fabrics are made of polypropylene or polyester.

Woven geotextiles are constructed of interlocking warp and fill filaments much like that of clothing fabrics. Various weave patterns are acheived by the method by which the filaments are interlaced. For this study, the geotextile product tested consists of a multi-layer slit-film woven fabric sewn together by polyester thread in the warp direction.

Nonwoven geotextiles are of either stapled or continous extrusion construction. Stapled products are made from the by-product of trimming the edges of nonwoven geotextiles. The three to six-inch long pieces of polymeric thread are remanufactured into a product very much like the continuous extrusion nonwoven, but having slightly lower strength properties and slightly higher hydraulic properties. Both stapled and continuous extrusion nonwovens are typically bonded by mechanical, chemical or thermal methods.

Needle-punching is the most common method of mechanical bonding. After the nonwoven geotextile is extruded, it is passed through a series of reverse-hooked needles that pull fibers back through the fabric as the needles are retracted. Thermal bonding consists of melting the nonwoven geotextile after the initial extrusion. Heat calendering, a form of thermal bonding, is often used on both stapled and continous extrusion nonwovens to improve clogging resistance and construction workability. Heat calendering consists of passing the nonwoven through smooth heated rollers. The result is a product with a stiffened-bordy consistency on one or both sides. From a construction standpoint, this "bordy" product is much easier to place.

Geogrids are predominantly produced from polypropylene, polyethylene and polyester polymers. The two most common methods of geogrid construction are that of drawing and welded-overlap.

For drawn grids, the polymer is extruded in molten form into sheets which are then perforated, or extruded through a die to form the grid aperatures. The sheets are then mechanically stretched as the polymer cools. This stretching process acts to form the desired geometry of the grid, but most importantly to align hydrocarbon chains within the polymer resulting in increased tensile strength, ductility and workability. The advantage of this method is a continuous product with junction strengths higher than that of the grid members themselves.

A second method of construction is that of welded-overlap. Individual strands of polymer in the warp and fill directions are extruded and drawn to the desired dimension and consistency. The strands are then welded or glued together to form the desired grid geometry. The advantage of welded-overlap construction is that custom grids can be produced having direction-specific properties. The disadvantage of this method is considerably lower junction strengths due to the joint bonding process.

The magnitude of the shear strength developed at the soil/geosynthetic interface is a function of the degree of interlocking between the soil and the geosynthetic. Geogrids, due to their open nature, result in higher shear strength produced by passive resistance during deformation. Strength of the junctions is critical since the majority of shear resistance is developed as a result of interaction between the cross or fill direction members. Grid geometry, therefore, is an important component in interaction behavior (7).

Geometry and Surface Roughness

The physical shape and texture of a geosynthetic greatly influences its interaction behavior with a soil. Comparing the behavior of a slick HDPE geomebrane with that of a geogrid illustrates this point. The open nature of the geogrid allows it to interlock with the soil while the slick geomembrane develops a plane of weakness where sliding may occur at relatively low shear stresses. Geotextiles also exhibit some degree of interlock depending on their surface texture and roughness.

The surface roughness is primarily a function of the depth of the weave pattern on the surface of the geotextile, the spacing and type of fibers, the type of weave, and the stiffness of elastic modulus of the fibers.

Interface Friction Parameters

The interface friction parameters for a soil/geosynthetic system may be evaluated by direct shear or pullout testing. The development of interface friction generally results from the mechanisms of rolling, sliding and dilation between soil particles and geosynthetics. By measuring shear resistance at different values of normal stress, a Mohr failure envelope for the soil/geosynthetic may be plotted. A plot of maximum shear strength versus normal stress similar to Figure (3-4) can be developed for a given system. The slope of the envelope is given by the angle delta and is termed the interface friction angle. The shear stress intercept is the apparent cohesion of the system and is termed the adhesion, a.

Tensile Elastic Modulus

The stress strain relationships for polymeric materials are highly dependent on boundary conditions and strain rate. Three moduli are commonly used to define elastic modulus: initial tangent, offset and secant (Figure 2-3). In addition to the aformentioned variables, the elastic modulus is a function of bonding, and the method and type of confinement.

Confined Tensile Strength

Comparing unconfined wide width test results with that of confined pullout tests, for many products, is like comparing apples and oranges. Granted, soil confinement does not make a product stronger, but confinement does make the soil/geosynthetic system stronger in many cases. In contrast, consider a geogrid used in a highly angular stone fill. Confined tensile testing can result in values of strength lower than that obtained from wide width tests; Frankenburger, (1987). Geosynthetics act as the transfer mechanism for tensile forces developed in a structure to pass into the soil. Confined tensile strength can be evaluated by several methods including triaxial cell, direct shear and pullout tests. Figures (3-5) and (3-6) give schematic views of these test methods.

The mechanical and physical properties of polyester and polypropylene fibers are summarized in Table 3-2.

EXTERNAL FACTORS

External factors can greatly influence the behavior and performance of a soil/geosynthetic system. Enviromental factors such as the type of pore fluid, temperature, ultraviolet radiation exposure and anticipated life span should be considered in the proper design a structure or facility.

Pore Fluid Type

In the case of a hazardous waste impoundment facility, the influence of pore fluid can be extreme. Chemical compatability of a geosynthetic with a given pore fluid can be evaluated by means of a modified EPA 9090 test in which index tests are performed on a geosynthetic before and after exposure to an anticipated pore fluid.

Seepage forces are also of great concern. Water tends to flow along a soil/geosynthetic interface and can cause migration of fines, leaving voids which may lead to localized collapse or failure if proper drainage is not provided.

Ultraviolet Exposure

Ultraviolet (UV) radiation can cause accelerated degradation of polmers used in the manufacture of geosynthetics. Polypropylene and polyethylene have the lowest UV stability while polyester has a greater resistance (3). Admixtures such as carbon black are typically used to improve U.V. resistance. In the case of large impoundment facilities where several acres of a geomembrane may be exposed during construction, UV radiation from natural exposure to sunlight can cause premature degradation if scheduling is not expedient. Care should be taken before and during construction to keep stockpiled geosynthetics properly protected.

Temperature

Hand in hand with UV exposure is the issue of temperature. On large projects, excessive heating of the geosynthetics may cause large elongations. Seaming can be a nightmare when daily temperature fluctuations are excessive. Tearing of seams or excessive wrinkles may result. Ideally, large projects should be constructed during seasons of mild temperature. In addition to these construction precautions, all laboratory testing of geosynthetics should be carried out with field temperatures in mind. Impact testing of geogrids used for snow fence in the Artic region would obviously require lab temperatures some what lower than the anticipated in-situ temperature.

Construction Methods

Probably the single most important factor in achieving a geotechnical structure that functions properly and safely is construction practice. Untrained personel, poor equipment, poor quality control and poor placement of geosynthetics may lead to unexpected deformations and failure.

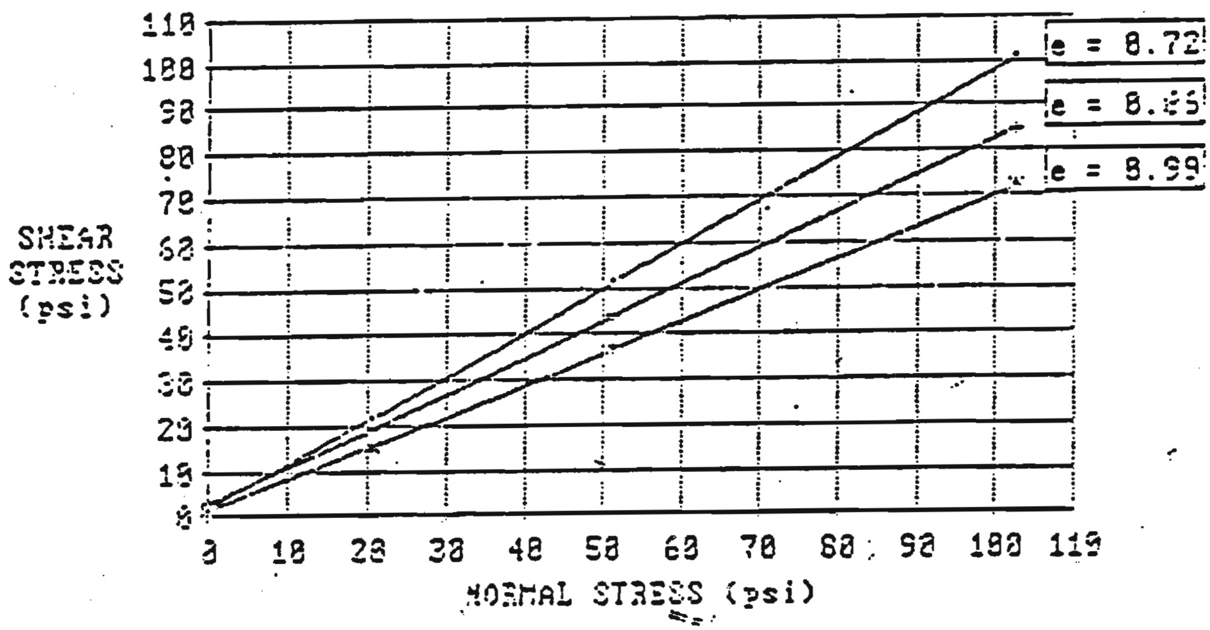


Figure 2-4. Effect of Void Ratio on Friction Angle and Shear Stress for a Concrete Sand (after Newman, 1986).

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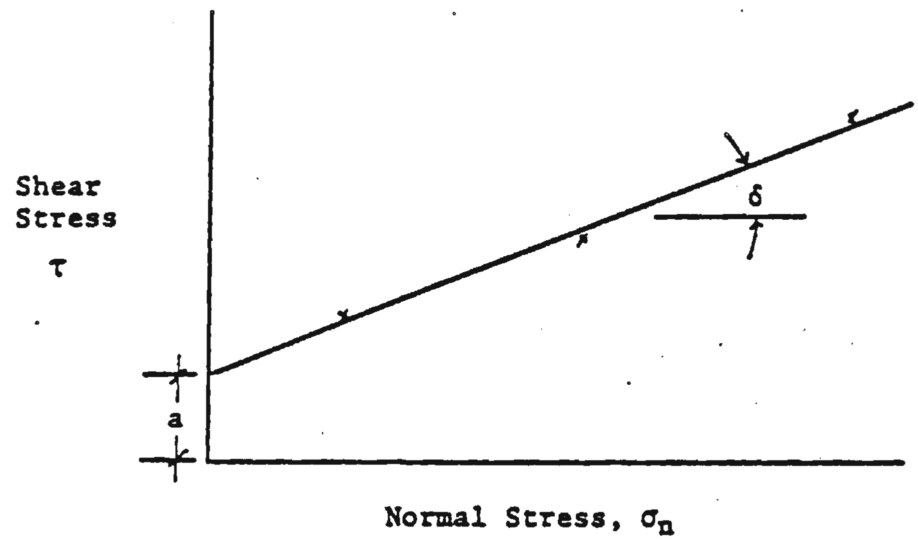
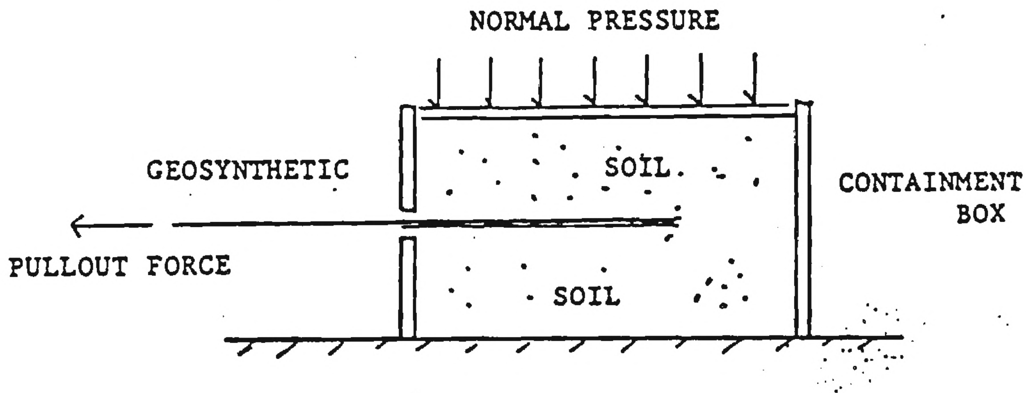
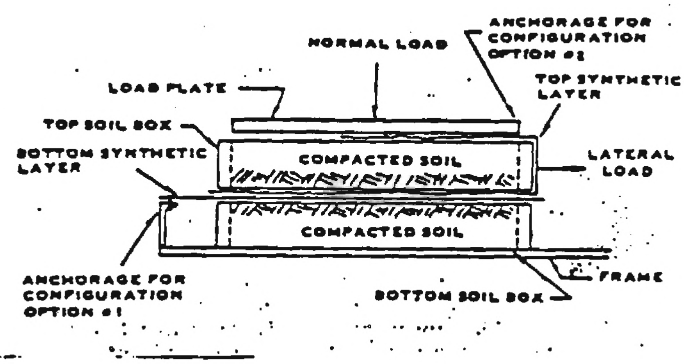


Figure 3-4 Failure Envelope for Geosynthetic Reinforced Soils (after Newman, 1986).



a. PULLOUT TEST DEVICE



b. DIRECT SHEAR TEST DEVICE (After Williams and Houlihan, 1986)

Figure 3-5 Test Methods for Determining in-Soil Strength Characteristics of Geosynthetics (after Newman, 1986)

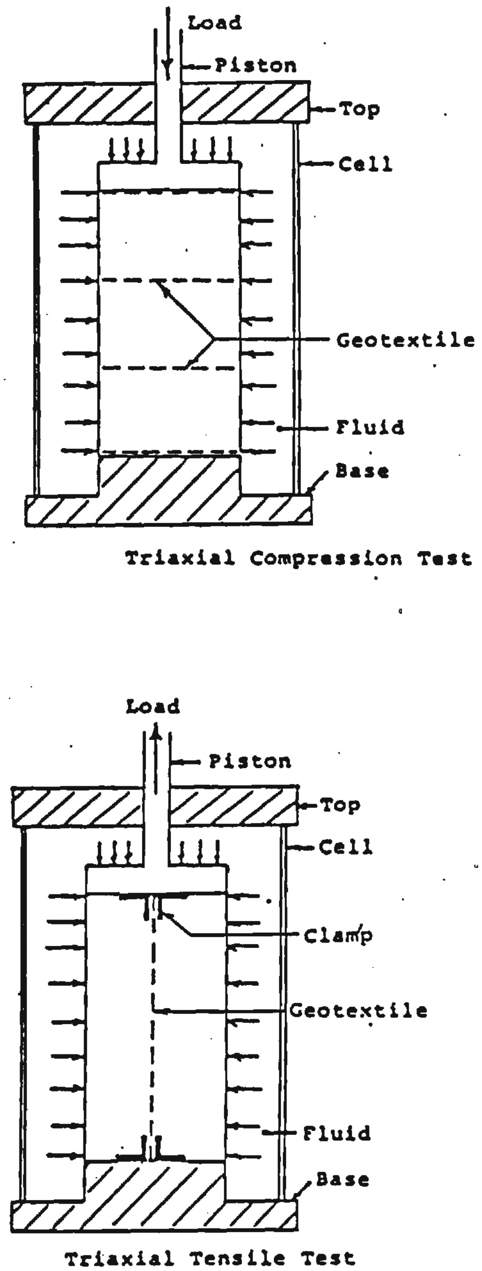


Figure 3-6. Triaxial tests modified to analyze in soil geotextile strengths (after Christopher et al., 1986).

Table 3-2

Relative Polymer Fiber Properties (after Lai, 1986)

Property	Relative Value*	
	Polypropylene	Polyester
Cost	2	4
Specific Gravity	2	4
U.V. Stability**	2	4
Thermal Stability	2	4
Chemical and Biological Stability	2+-4	3-4
Strength	2-4	4
Failure Elongation	4	2
Elastic Modulus	2	4
Creep Susceptibility	4	2
Impact Cutting Resistance	4	2

* Comparison for fibers currently (1979) common in geotechnical fabrics. The scale is 4 Highest - 3 Slightly Lower - 2 Considerably Lower. It is important to note that a high rating is not necessarily desirable. Also, in many instances the fabric properties will be controlled more by the fabric construction than by the polymer fiber.

** Unstabilized.

CHAPTER 4

DIRECT SHEAR TESTING PROGRAM

Scope

Direct shear analyses were performed to evaluate the interface friction properties between the stitchbonded geotextiles and two types of soil. The analyses were performed parallel and perpendicular to the stitch lines at confining stresses of 100, 250 and 500 psf. A total of 16 analyses were performed.

Two types of soil were used in the analyses; Ottawa 20/30 sand and a sand-bentonite mixture. Ottawa 20/30 sand is a quartzitic, rounded to well rounded, highly uniform sand, with a uniformity coefficient of 1.3 and a USCS classification of SP. All of the sand passed the No. 20 sieve and was retatined on the No. 30 sieve. The sand was compacted to a density of 98 pcf, with a relative density of approximately 95%. The soil was placed, compacted and tested in a dry state (34).

The clay-sand mixture consisted of 90% Ottawa 20/30 sand and 10% bentonite. The PI was 27 and the Modified Proctor maximum dry density was 119 pcf at 12.6% moisture content. The void ratio at time of testing was 0.64 and the degree of saturation was 62%. This soil was compacted at optimum moisture content plus 2% to a dry density of 115 pcf (34).

Methodology (Equipment setup/procedure)

The modified direct shear device used in this study was developed at the Georgia Institue of Technology (33). The device has a sample plan area of 12 in. X 12 in. which allows for a large amount of deformation and also the ability to test complex composites. Figure (4-1) gives a schematic view of the modified direct shear device.

The device consists of an upper and lower box filled with soil. The geosynthetic specimen is placed between the two boxes of soil and a normal load applied. The lower box is held stationary while the upper box is loaded horizontally. The geotextile may be loosely placed between the soil layers and allowed to slide on the plane of minimum resistance or restrained at one end to force sliding along the upper surface. Because of the slow build-up of stress, the peak stress and static coefficient of friction; as well as the residual stress and dynamic coefficient of friction can be measured.

For this study, various stitch bonded geotextiles were allowed to slide along the plane of minimum resistance. For each test performed, the following procedure was carried out:

1. Soil to be used was brought to the proper moisture and well mixed.
2. The lower box of the direct shear device was filled with soil and compacted to the required density.
3. The geosynthetic to be tested was placed center of the lower box.
4. The upper box was placed on top of the geosynthetic and step 2 repeated.
5. The normal load plate was placed on top of the upper box and the loading yoke and normal load cell positioned.
6. The horizontal dial indicator was attached to the rear of the upper box.
7. The test was iniated. The desired normal load was applied using the system pressure regulator. The proper horizontal rate of deformation was set using the horizontal flow regulator.
8. Initial values from the dial gage, normal load cell and horizontal load

cell were recorded on the data sheets;

9. The horizontal flow valve was then turned on to initiate the test sequence.

10.Values of horizontal movement and horizontal load were recorded at one minute intervals. The normal load was adjusted throughout the test to maintain a constant normal stress. The normal load was typically decreased as the soil dilated and then increased slightly as a peak was reached.

11.The test was continued until three peaks were observed; and

12.At the end of a test, the system pressure was released and the upper box retracted. At this point, a new normal stress was applied and the test repeated or a new series of tests iniated.

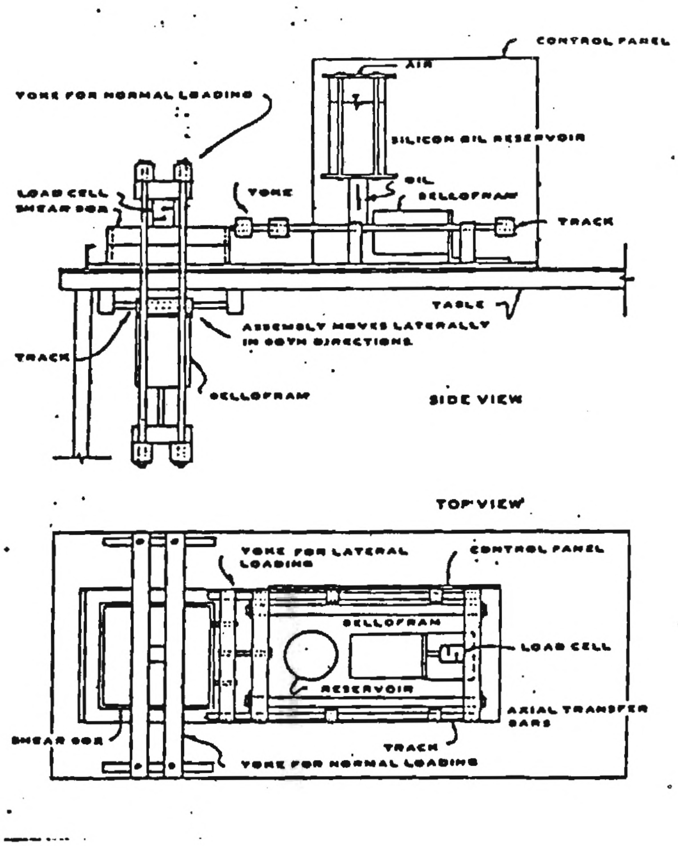
Results

Results for the direct shear program are presented in tabular and graphic form The results of analyses are summarized in Table (4-1) and Figures (4-2) through (4-15) in Appendix (B). The effects of product type, stitching orientation, and clay content on the interface friciton parameters (u,delta,a) are apparent. The largest values of u,delta and a were seen in the woven/nonwoven composite product. The high effective surface area and greater roughness of the nonwoven geotextile as opposed to the woven slit-film geotextile results in higher interface friction parameters.

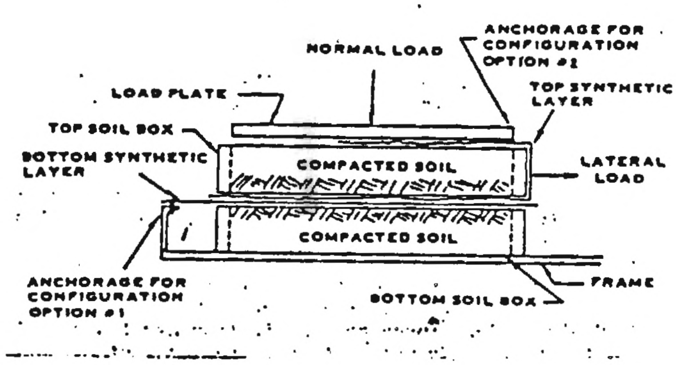
For the GTF products 450, 500 and 800 (1,3 and 4 layer), values for the coefficient of friction, u, ranged from 0.33 to 0.48. Values for the interface friction angle, delta, ranged from 21 degrees (GTF 450 in SP) to 25-26 degrees(GTF 500 & 800 in CL).The effect of stitching orientation was found to be relatively minor.

In all cases, the adhesion decreased noticeably for the sand-bentonite mixture. However, an increase in clay content generally increases a soil's cohesion and therefore increases shear strength. This decrease may be attributed to the presence of the clay at optimum moisture content serving to lubricate the soil-geosynthetic interface thereby decreasing the apparent shear strength in that zone. Bulking can not be used to explain the higher values of adhesion for the clean dry sand analyses, since by definition, bulking is a phenomena of moist sands.

The values of u were found to decrease with the increase in clay content for all cases except the woven/nonwoven composite. As mentioned earlier, this increase in interface friction is likely due to the increased surface area and surface roughness of the nonwoven geotextile.



(a) Profile and plan view



(b) Specimen configuration

Figure 4-1 Modified Direct Shear Device (after Williams and Houlihan, 1986).

TABLE (4-1)

DIRECT SHEAR TEST RESULTS

Product	Soil/Product Configuration	Friction Coef. (-)	Inter.Frict. angle, (Degrees)	Adhesion, a (PSF)
GTF 450 (single layer)				
	Ottawa Sand			
	* Parallel-	---	---	---
	* Perpendicular-	---	---	---
	Clayey Sand			
	Parallel-	0.38	21	12
	Perpendicular-	0.42	22.7	13
GTF 500 (3 layer composite)				
	Ottawa Sand			
	Parallel-	0.43	23	40
	Perpendicular-	0.48	26	43
	Clayey Sand			
	Parallel-	0.33	18	32
	Perpendicular-	0.47	25	14
GTF 800 (4 layer composite)				
	Ottawa Sand			
	Parallel-	0.43	23	35
	Perpendicular-	0.47	25	38
	Clayey Sand			
	Parallel-	0.40	22	19
	Perpendicular-	0.39	21.5	28
GTF woven/nonwoven composite				
	Ottawa Sand			
	Parallel-	0.50	26.5	49
	Perpendicular-	0.49	26	51
	Clayey Sand			
	Parallel-	0.57	29.6	24
	Perpendicular-	0.61	31.4	22

NOTE:

1. Parallel and perpendicular refer to the direction of shear with respect to stitching.
2. Data marked by * was not performed in this study.

Conclusions (product specific)

For the four products tested the following comparative conclusions can be made. The single layer product, GTF 450, was found to have the lowest values of interface friction parameters. The three and four layer composite products (GTF 500 & 800) compare closely for values of coefficient of friction and internal friction angle in the clean Ottawa sand. The GTF 500 gives slightly higher values of apparent adhesion for both the clean sand and sand-bentonite clay mixture. The woven/nonwoven composite product was observed to have the highest values of interface friciton parameters of the four products tested. The increase is due to the presence of the nowoven layer of the composite.

Suggestions for Improvement

For this study, diect shear tests performed with the clean concrete sand used in the confined tensile strength testing program would have been helpful in order to make a more accurate comparison between the two testing programs. In addition, it would be desirable to perform direct shear tests with one end of the specimen held stationary. This again, would be helpful in comparing direct shear results with that of confined tensile strength testing.

CHAPTER 5

CONFINED TENSILE STRENGTH TESTING PROGRAM

Scope (product/soil specific)

The purpose of the confined tensile strength testing program is to establish a relationship between confining stress and embedment length for the GTF 800 product. In addition, it is desirable to make a comparison between confined tension test results and direct shear test results. This is done in an effort to determine how boundary effects and confinement influence values of interface friction parameters.

A total of seven tests were performed in a clean concrete sand at three confining stresses and four embedment lengths. This minimum number of tests is necessary to establish the confined tensile strength, (rupture), and to define a relationship to predict the mode of failure (rupture or pullout) of GTF 800 at a given confining stress and embedment length. All tests were performed with the product stitching perpendicular to the direction of loading.

Methodology (equipment setup/procedure)

The device used for the confined tensile strength program was developed at the Georgia Institute of Technology by Scott Newman as part of his master's degree program. Hereafter, the device will be refered to as the Large Scale Pullout/Creep Device (LSPCD). Figures (5-1P) and (5-2P) are photographs of the LSPCD in an operational mode. Figures (5-3) through (5-6) give schematical views of the LSPCD pressure system, clamp profile, and data aquisition system.

Figure (5-5) shows the clamp profile used for this study. Since the clamp is part of the soil-geosynthetic system being tested, it is necessary to account for this contribution. Figure (5-7) shows the shear stress mobilized on the clamp as a function of displacement. As part of the data reduction sequence, the value of clamp shear stress is subtracted from the total measured shear stress. Figure (5-8) shows a failure envelope for the clamp.

Geosynthetic Preparation

The sample preparation procedure is extremely important in order to assure the sample does not undergo undo strain at the clamping point. The sample preparation procedures are summarized below.

(1) The geosynthetic is cut to the desired length of embedment plus 18 inches to allow for the clamp

(2) Four pieces of TREVIRA 1120 nonwoven geotextile are cut; two 15 1/2 by 18 inch sections and two 3 by 15 inch strips.

(3) A 16 guage, 16 by 18 inch steel plate is placed in the location desired for curing of the epoxy reinforced section. The plate is covered with a large sheet of PVC plastic sheeting to prevent epoxy from bonding to the steel plate.

(4) Materials are arranged. Protective gear including rubber gloves and eye protection are worn. Ambient temperature is recorded.

(5) 125cc of the epoxy is mixed with an equal portion of the curing agent in a large tin. Thorough mixing of the two agents is required.

(6) One of the large sheets and small strips of the Trevira are impregnated with the epoxy mix. To ensure saturation of the epoxy into the nonwoven, the cloth is kneaded by hand.

(7) The large sheet of epoxy impregnated Trevira is spread onto the plastic

covered steel plate. The small strip is spread along the top end of the section to provide additional reinforcement.

(8) The impregnated nonwoven is covered with the geosynthetic sample.

(9) Steps (5) and (6) are repeated and the cloth is spread over the geosynthetic. Again, the strip is placed to reinforce the bolt hole location.

(10) The specimen is covered with a plastic sheet and another 16 guage steel plate. Approximately 20 to 30 pounds are placed over the steel plate to confine the reinforced section.

(11) After curing for 24 hours, the specimen is trimmed with a band saw and 11/16-inch diameter holes are drilled to fit the clamping plates.

(12) The two C 4 X 5.4 clamps are bolted to the sandwiched section.

The curing time of the epoxy is temperature dependent. Since stress concentrations develop around the bolts, it is imperative that the reinforcement be fully cured. Ideally, a curing temperature of 75 degrees F. or higher is desirable.

Soil-Geosynthetic Placement/Compaction

Proper placement of the geosynthetic and soil is very important in that a uniform void ratio and density must be maintained in order to achieve repeatable results. An air-driven compaction foot and a manually operated drop hammer were used for the tests run herein.

Based on moisture-density data obtained from a previous study (Figure 5-10), the concrete sand was brought to the desired moisture content. The soil was then compacted in 3-inch lifts to a depth equal to the midline depth of LSPCD frame. The air-driven compactor was used for the bulk of the material with the manually operated drop hammer being used in corners and along edges. At this point, the reinforced geosynthetic specimen was placed in the LSPCD frame taking great care to maintain center alignment. This alignment is very important in that premature rupture or edge-tear can result from a misaligned specimen.

Soil is then added to the top half of the LSPCD frame in 3 inch lifts. It is recommended that compaction of the soil on top of the specimen begin at the clamped end, proceeding to the rear of the LSPCD frame. It was observed that the specimen would migrate out of the frame under compaction if this was not done. Once the frame is properly filled and compacted, the top is screed off by an amount equal to the air-bladder thickness to ensure uniform application of normal stress. Soil is then molded into the corners to keep the bladder from being damaged.

Normal Load Application

Once the soil compaction sequence was completed, the top of the frame was vacuumed clean before placement of the rubber bladder. The cleaned rubber bladder was then placed on top of the frame and properly aligned over bolt holes. The top cover was then placed and bolted down with the A-325 bolts being torqued to roughly 105 foot-pounds. The normal stress pressure hose was then connected and the desired pressure applied. At this point, the system was checked for leaks by observing the pressure gage and by listening for leakage around edges. If a leak was detected, the bolts in that area were retorqued until the leak stopped.

Clamp Assembly Placement

With the LSPCD frame properly sealed, placement of the clamp assembly proceeded. The hydraulic cylinders were extended enough to clear the specimen clamp and the primary yoke was then pin-clip connected. The secondary yoke was then slipped into place through the primary yoke and connected by two 5/8-inch bolts. At this point, the hydraulic cylinders were extended to give about 1/8-inch clearance between the load cell and secondary yoke button. Two flat washers were placed under the secondary yoke to hold it on the centerline of the primary yoke. Extreme care was taken to make sure the two hydraulic cylinders were equally extended. If this were not done, premature rupture or edge-tear would result.

Data Aquisition Hookup

The LVDT was mounted onto the specimen clamp and lightly tightened. Over-tightening the LVDT clamp may cause damage to the instrument or give extraneous results. The leads from the LVDT and load cell are then connected to the data aquisistion system, consisting of a signal conditioner, amplifier and IBM personal computer. With power on, the load cell and LVDT are adjusted to their proper limits.

At this point, the testing sequence was initiated The software (Frankenburger, 1987) was loaded on the PC, Appendix (A), and information concerning specimen width, sample interval and type of test were inputted.

The confined tensile stength test may be performed either by stress or strain control. For this study, stress control was used with a loading rate of approximately 150 lbs./min. This rate resulted in deflection rates varying between 0.05 and 0.10 inches per minute. The load was applied to the system by controlling a pressure regulator mounted in-line with a compressed nitrogen source. The nitrogen gas was used to drive the system. The load was continually applied until the specimen failed by either rupture or pullout.

Test results were stored on floppy disk and included values of horizontal deflection, total load, alpha (load per unit width), and elapsed time. The stored data is typically reduced in mumber of data points due to the enormous amount of data that can be accumalated. This was particularly true of the geotextile tested in this study. At the deformation rates used in the analyses, geotextiles generally elongate on the order of ten percent before rupture.

A Fortran program, "CLAMPG.FOR," was used to subtract off the contribution of the clamp. Figure (5-7) gives the clamp calibration used for this study. The data was then zeroed for the intial readings at time zero. The reduced data was then transfered to a graphics system for the final presentation. For this study, MICROSOFT Chart was used to prepare the confined tensile strength graphs. A user's manual for the data reduction procedure and listings of the fortran programs are given in Appendix (A) (10).

Results are plotted as the horizontal stress, alpha, versus horizontal deflection of the specimen clamp. The horizontal stress, alpha is defined as the total load on the sample divided by the width in units of pounds per foot of width.

Shutdown Procedure

At the end of a testing sequence, pressure was vented from the oil reservoir and rubber bladder. The hydraulic cylinders were then be retracted to allow the removal of the clamp assembly. The frame top was then unbolted, and the top layer of soil removed. A drive cylinder sample of the soil was taken after each test.

The oil reservoir was designed for a maximum internal pressure of 650 psi with a safety factor of four. Over time the connections may loosen due to fatigue. Therefore, the bolts should be tested for the proper torque of 40 foot-pounds. The O-ring seals may be expected to deteriorate over time and should be immediately replaced if any leakage is detected. When servicing the pressure vessel, the general condition of the steel should be noted. Any corrosion or damage reduces the factor of safety (21).

Test Results

Soil Analyses

For the confined tensile strength testing program a light-tan, fine concrete sand (SP) was used which was obtained locally. The soil analyses presented here include: sieve analysis, relative density, standard proctor and direct shear testing (21).

The sand used is uniformly graded with a coefficient of uniformity of 3.1 having a maximum particle size of approximately 1 mm. The gradation curve shown in Figure (5-9).

Table (5-1) presents the results of relative density determination as per (ASTM D2049) performed on the sand. Minimum and maximum void ratios were determined as 0.652 and 1.09, respectively. This corresponds to a maximum dry density of 100.0 pcf and a minimum dry density of 79.3 pcf.

Standard Proctor results as per ASTM D698 are presented in Figure (5-10). The characteristic flat moisture-density curve shows that equal densities can be achieved over a wide range of moisture contents. This was quite an advantage for this testing program in that moisture content adjustments were seldom necessary.

Results of three direct shear tests performed on the concrete sand are presented in Figure (5-11) representing strength as a function of void ratio. Figure (5-12) shows the failure envelope for the concrete sand. Figures (5-13) through (5-15) are individual plots of Mohr envelopes, shear stress versus strain and shear stress versus volumetric strain for void ratios of 0.72, 0.86 and 0.99. The friction angles reported for these void ratios are 44.4, 39.5 and 35.0, respectively (21).

Compaction was verified at the conclusion of each test by the drive tube method. For the seven tests performed, dry density varied between 88 and 90 pcf. Table (5-2) lists the degree of compaction for each test as well as other test specific variables.

Pullout Tests

Results of the confined tensile strength tests are summarized in Table (5-2). The results are also given graphically in figures (5-16) through(5-19). Individual plots of pullout results are given in Appendix C. (Figures 5-20 through 5-26). The first test was performed with a full embedment length of 40 inches and a 10 psi confining stress. This was done in order to force a rupture failure, thereby establishing the ultimate confined tensile strength of the GTF 800. Holding the confining stress constant at 10 psi, three additional tests were performed with decreasing embedment lengths of 25, 20, and 15 inches, respectively. These three analyses resulted in a pullout mode of failure.

The results of the analyses are summarized in Figure (5-17). As the embedment length is increased, the pullout strength increases up to the point of rupture at high values of confining stress. A second series of three tests was performed holding the embedment length constant at 20 inches and varying the confining stress at 2, 5, and 10 psi. These results are summarized in Figure (5-16). As can be seen, a similar relation results. As confining stress is increased, the pullout strength increases.

The GTF 800 geotextile was observed to elongate between eight and ten percent for the rupture mode of failure and the loading rate used. In reaction to this large longitudinal movement, the geotextile was observed to "neck-down" considerably upon inspection of the failed specimen.

For the specimens that failed by pullout, quanitative measurement of strain was not possible. At the present stage of the LSPCD's development, the distribution of stress and strain along the length of the specimen can only be hypothesized. However, inspection of the pullout-failed speciemens suggests that for the range of confining stresses investigated, no appreciable amount of permenant deformation occured.

Photographs were taken of the failed specimens and are presented in Figure (5-28P).

The relations shown in Figures (5-16) and (5-17) of alpha versus horizontal deflection exhibit an elasto-plastic behavior that would be expected for this type material. In Figures (5-18) and (5-19), it can be seen that for horizontal deflections up to approximately 1.0 inch, this material obeys a constant relation for high confining stresses; regardless of the embedment length. Figure (5-18) shows this relation to be a straight line using a logarithmic plot.

From these two relations, the importance of confined tensile strength data for use in design may be realized. These analyses make it possible to predict the mode of failure based on the type of soil, confining stress and embedment length.

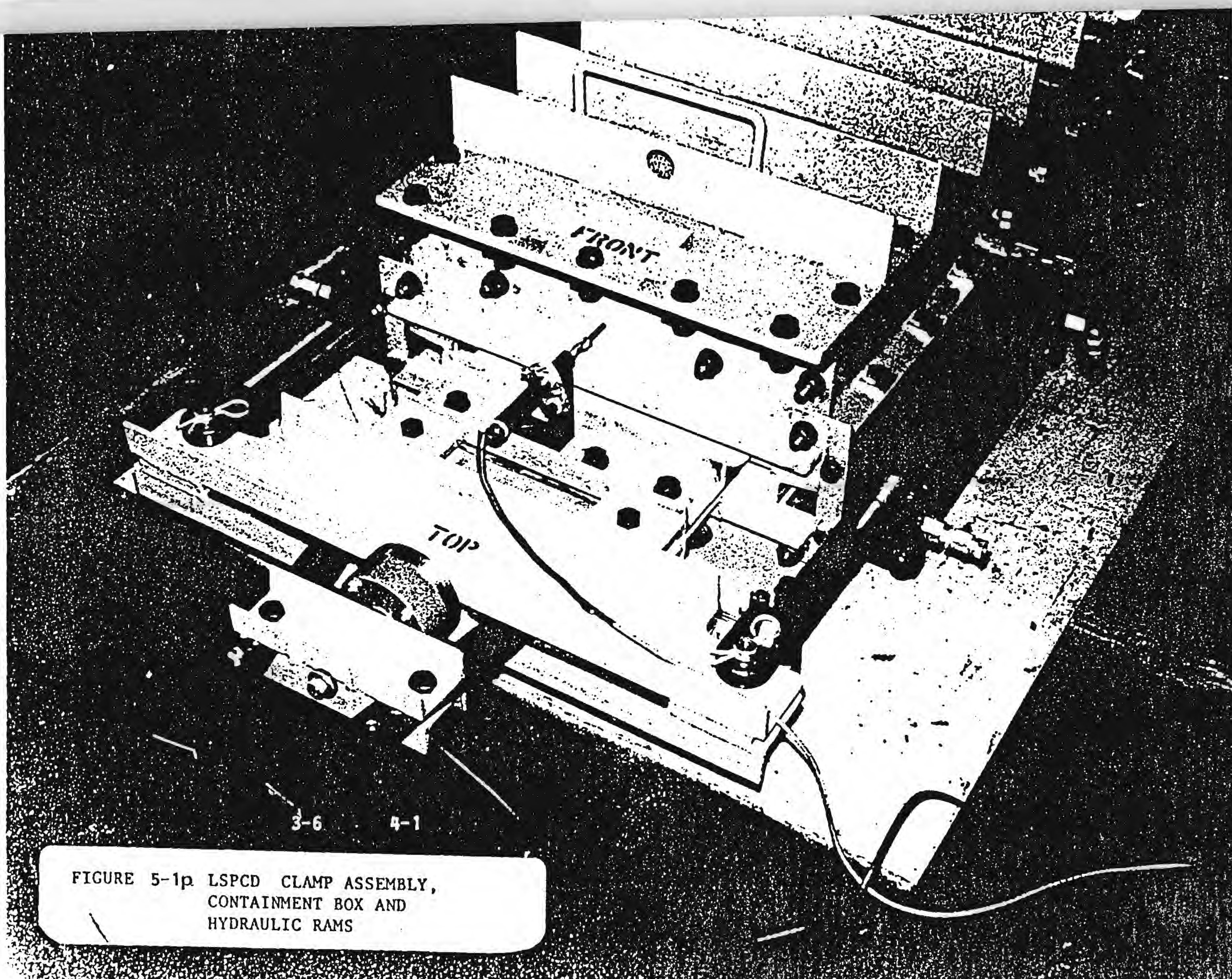


FIGURE 5-1a LSPCD CLAMP ASSEMBLY, CONTAINMENT BOX AND HYDRAULIC RAMS

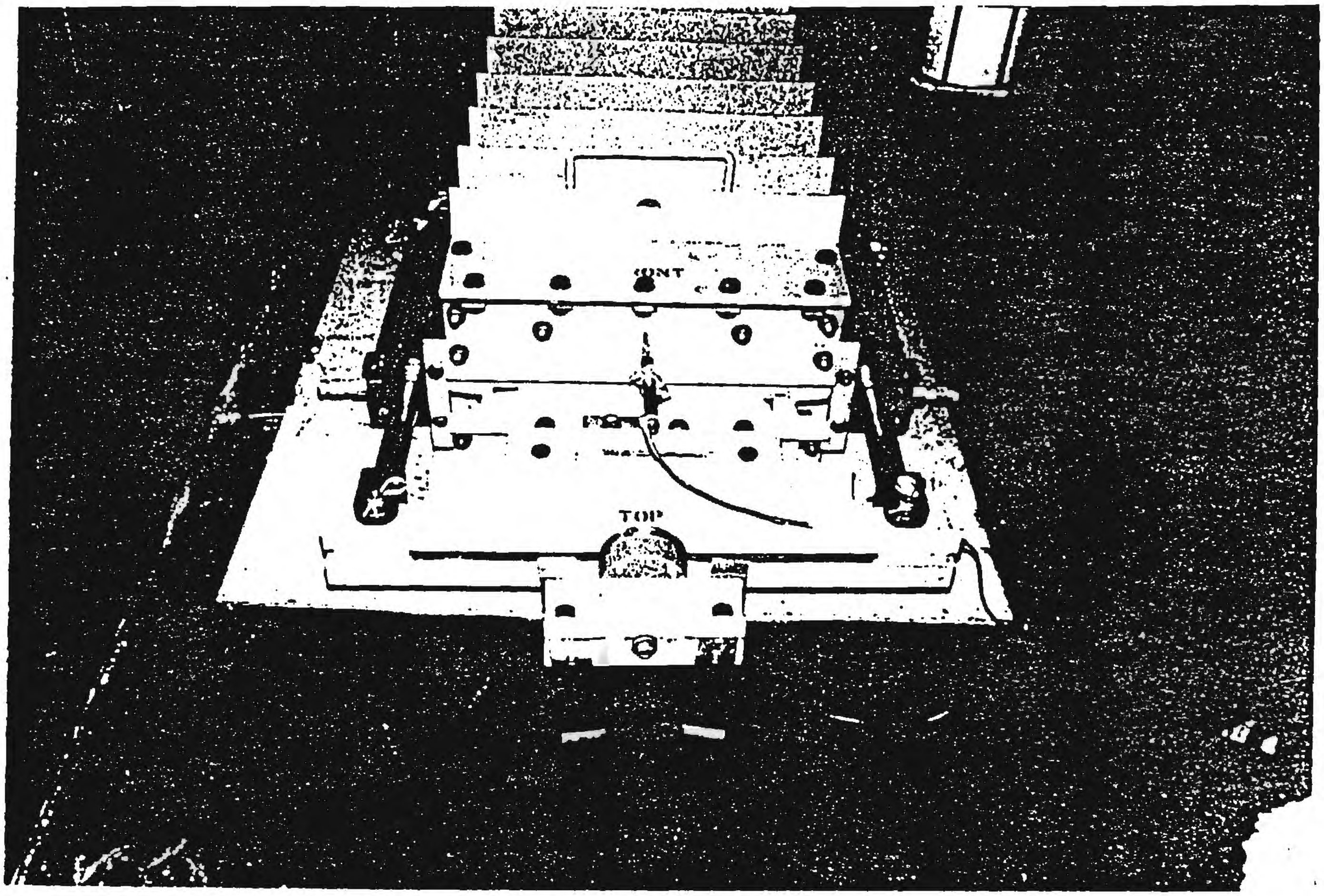


FIGURE 5-2P . PHOTOGRAPH OF THE CLAMP ASSEMBLY

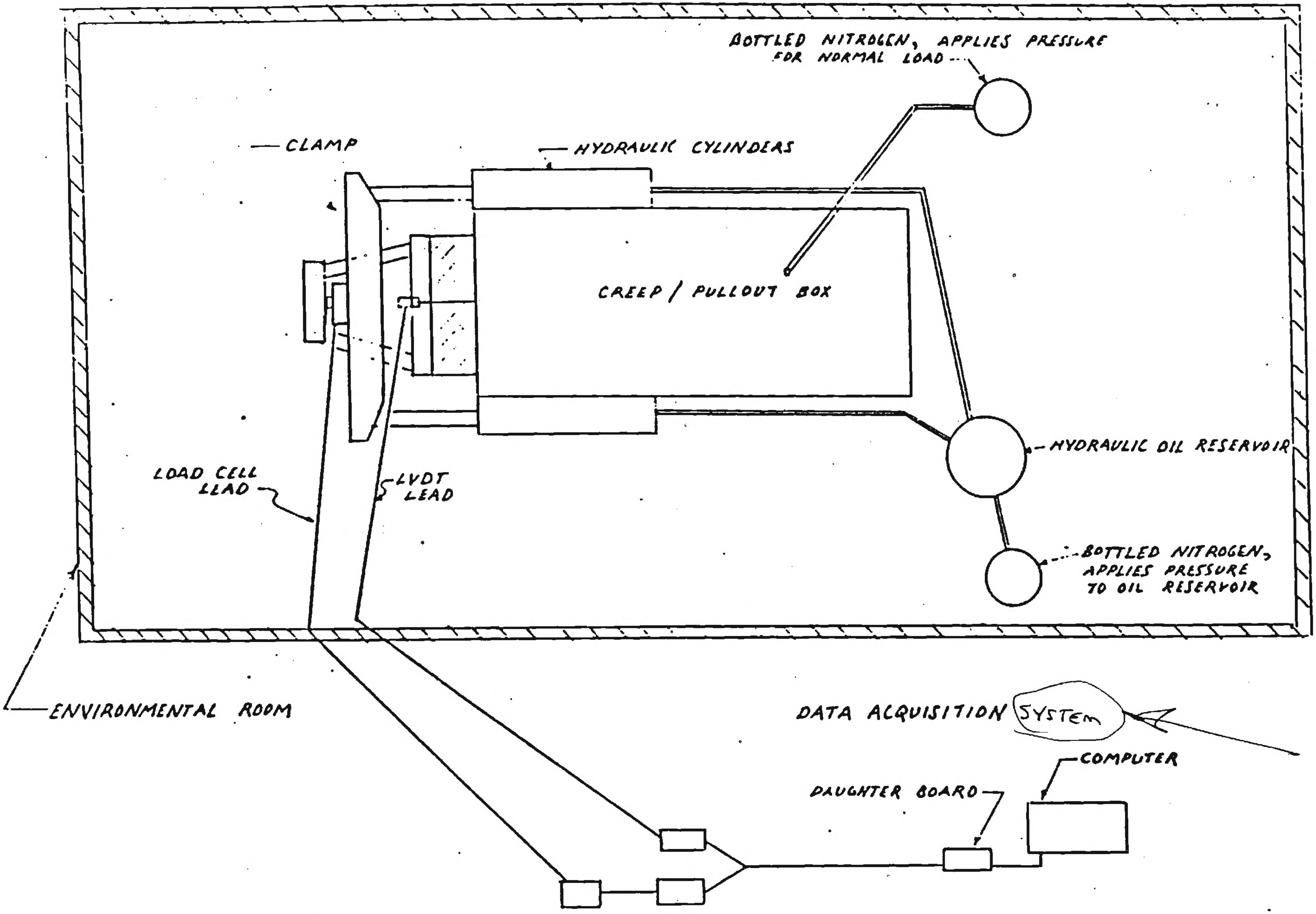
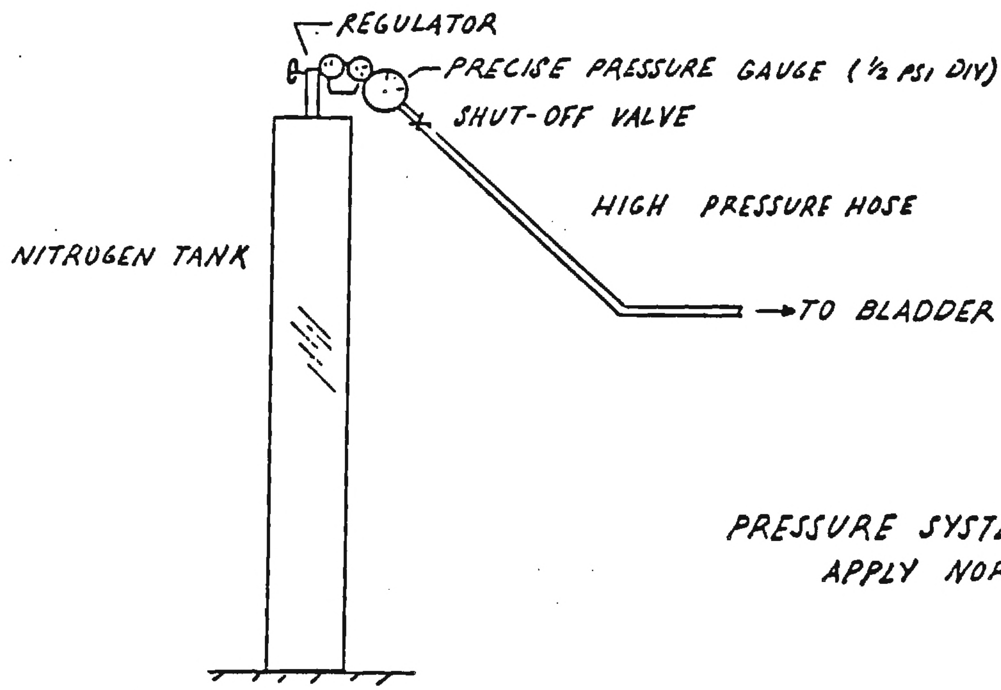


Figure 5-3 Large Scale Pullout Creep Device (after Newman, 1986).



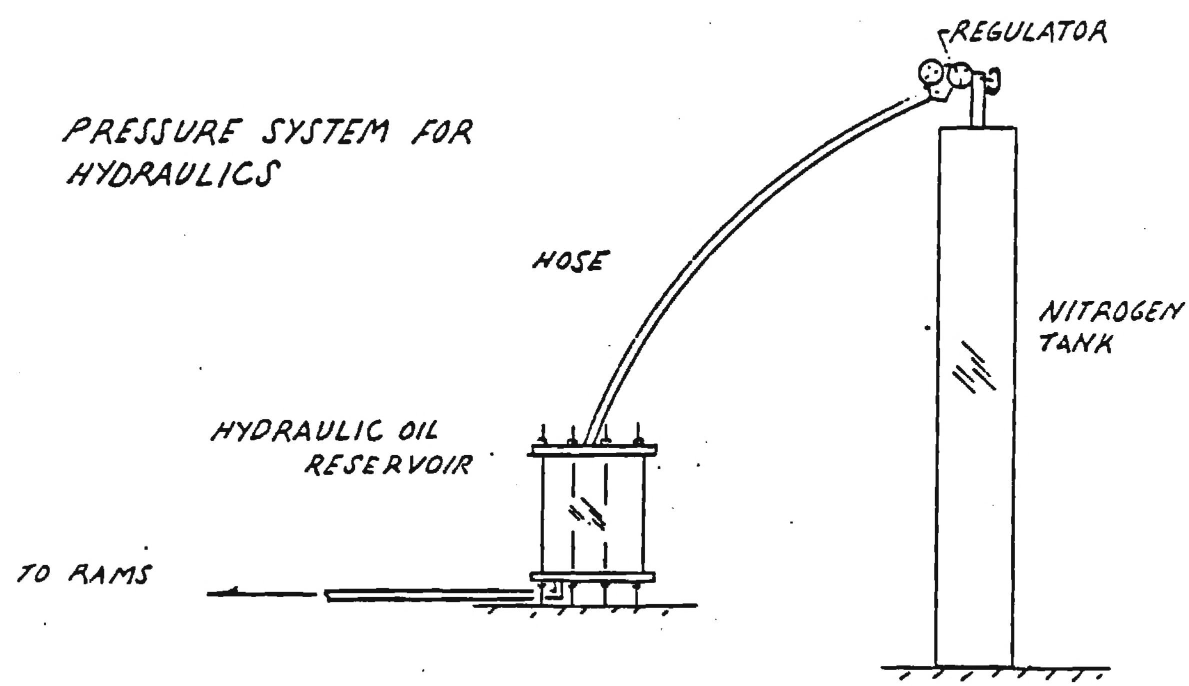


Figure 5-4 LSPCD Pressure Systems (after Newman, 1986).

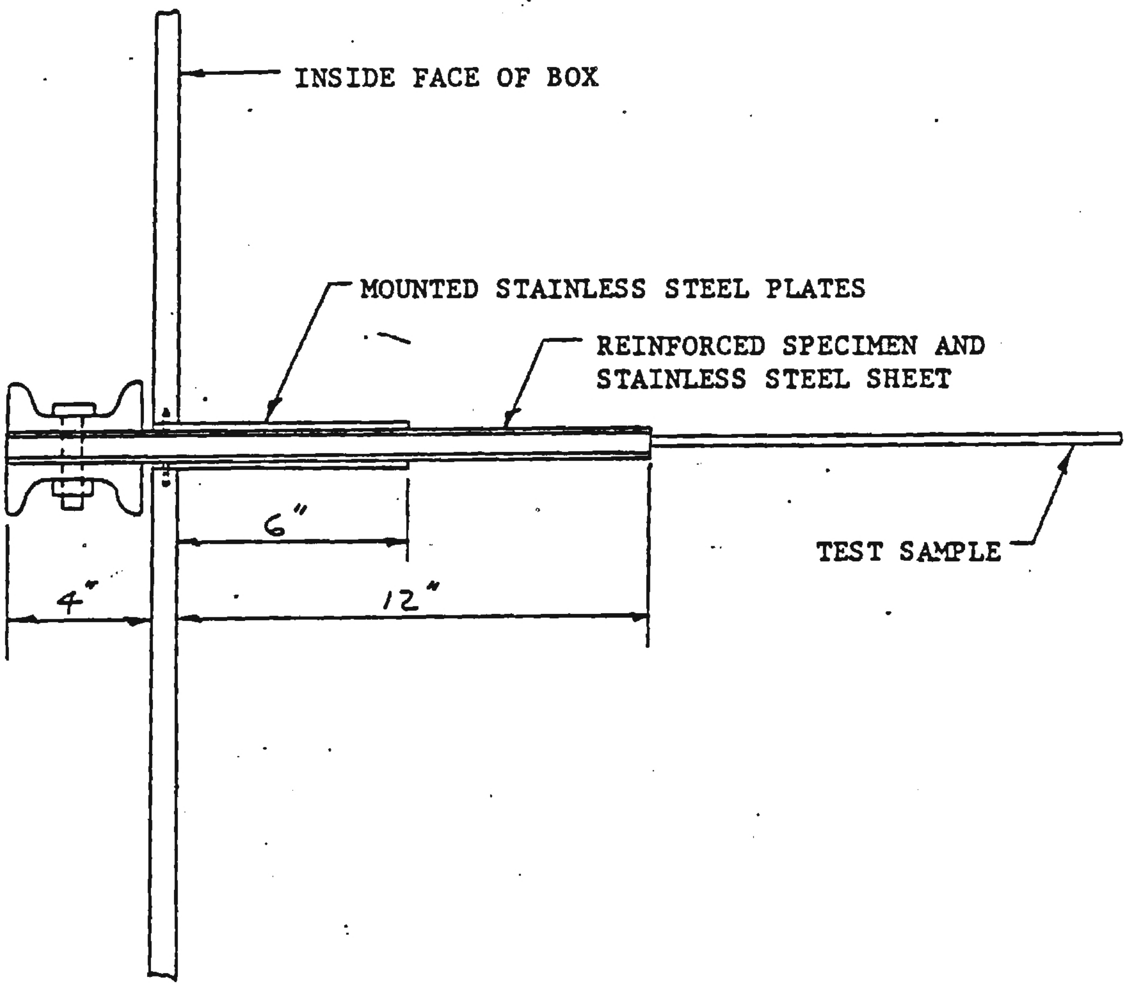


Figure 5-5 Clamp Profile Used in the LSPCD.

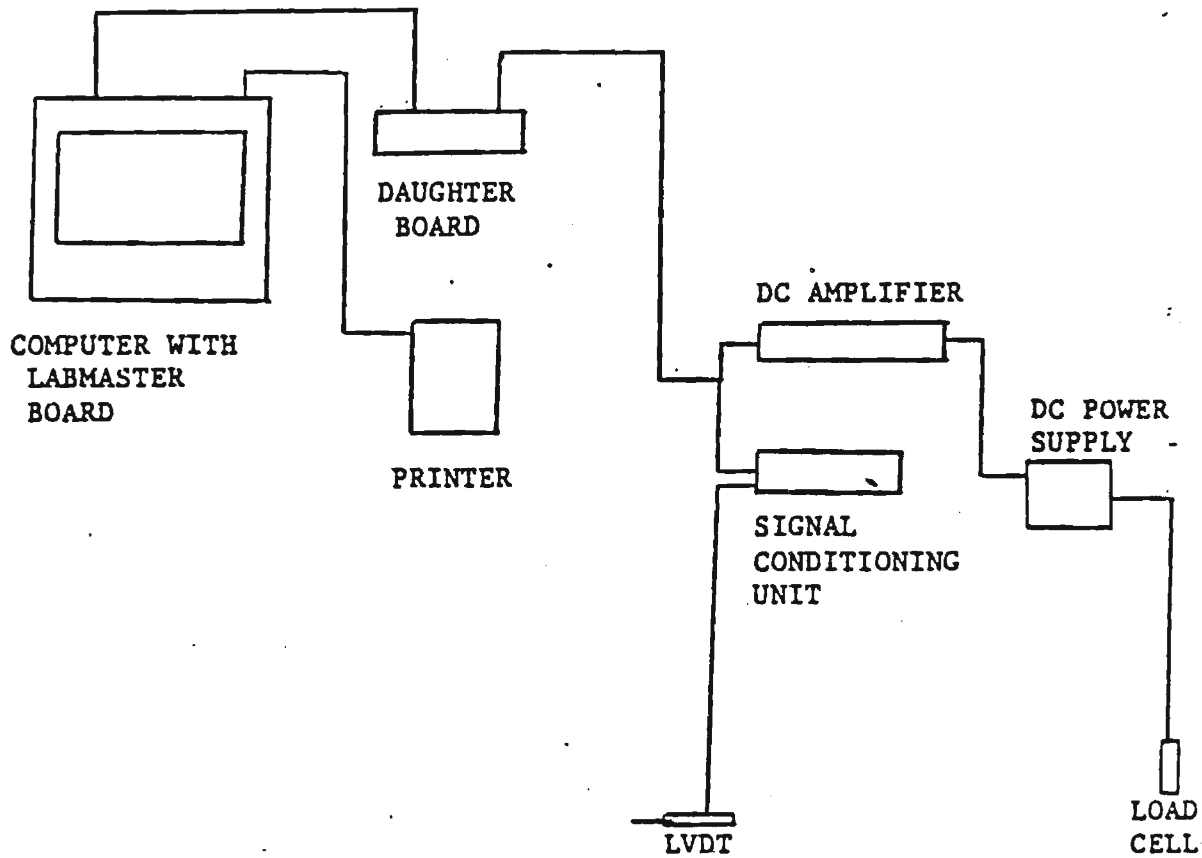


Figure 5-6 Data Acquisition System (after Newman, 1986).

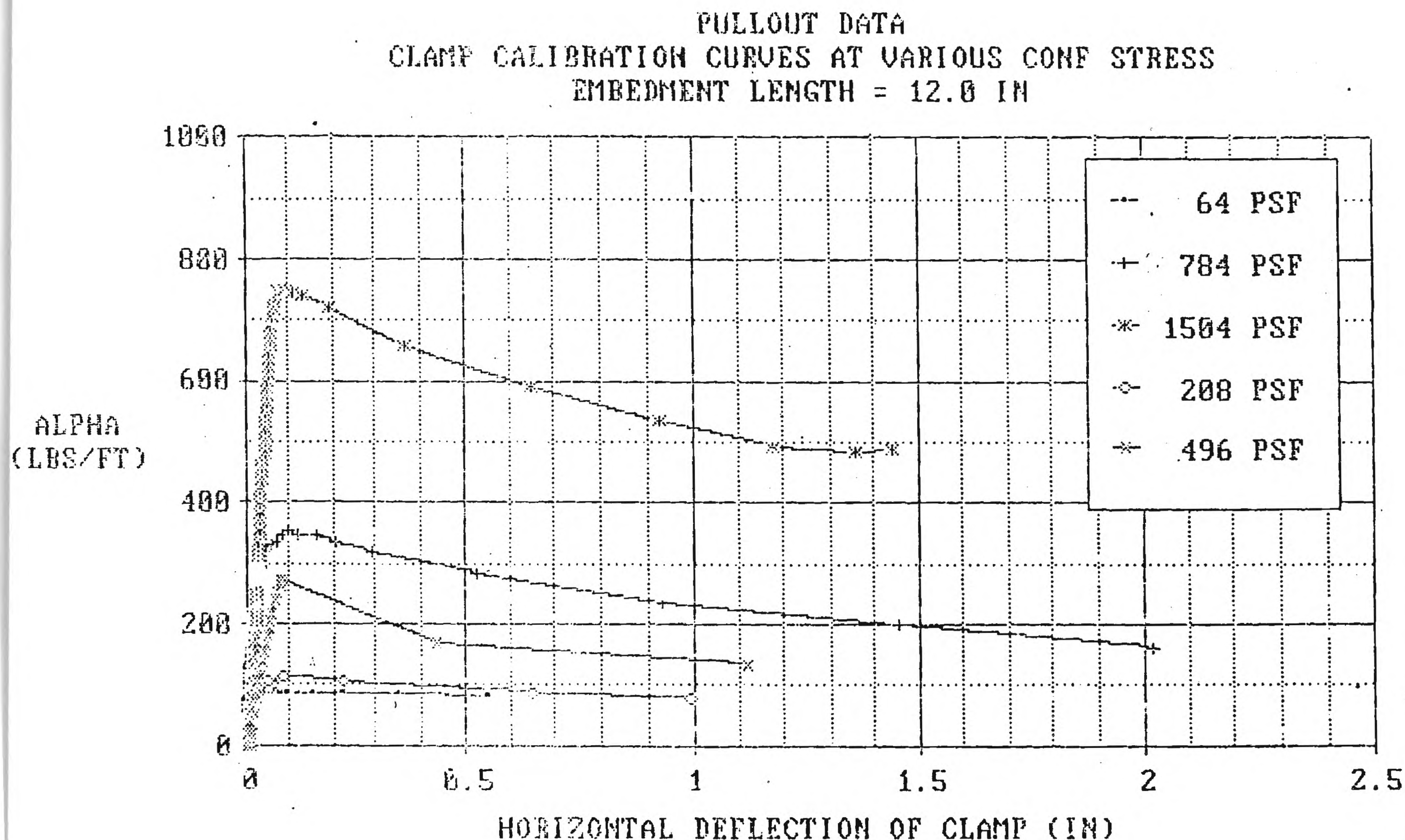


Figure 5-7 Clamp Calibration Curves for Various Confining Pressures

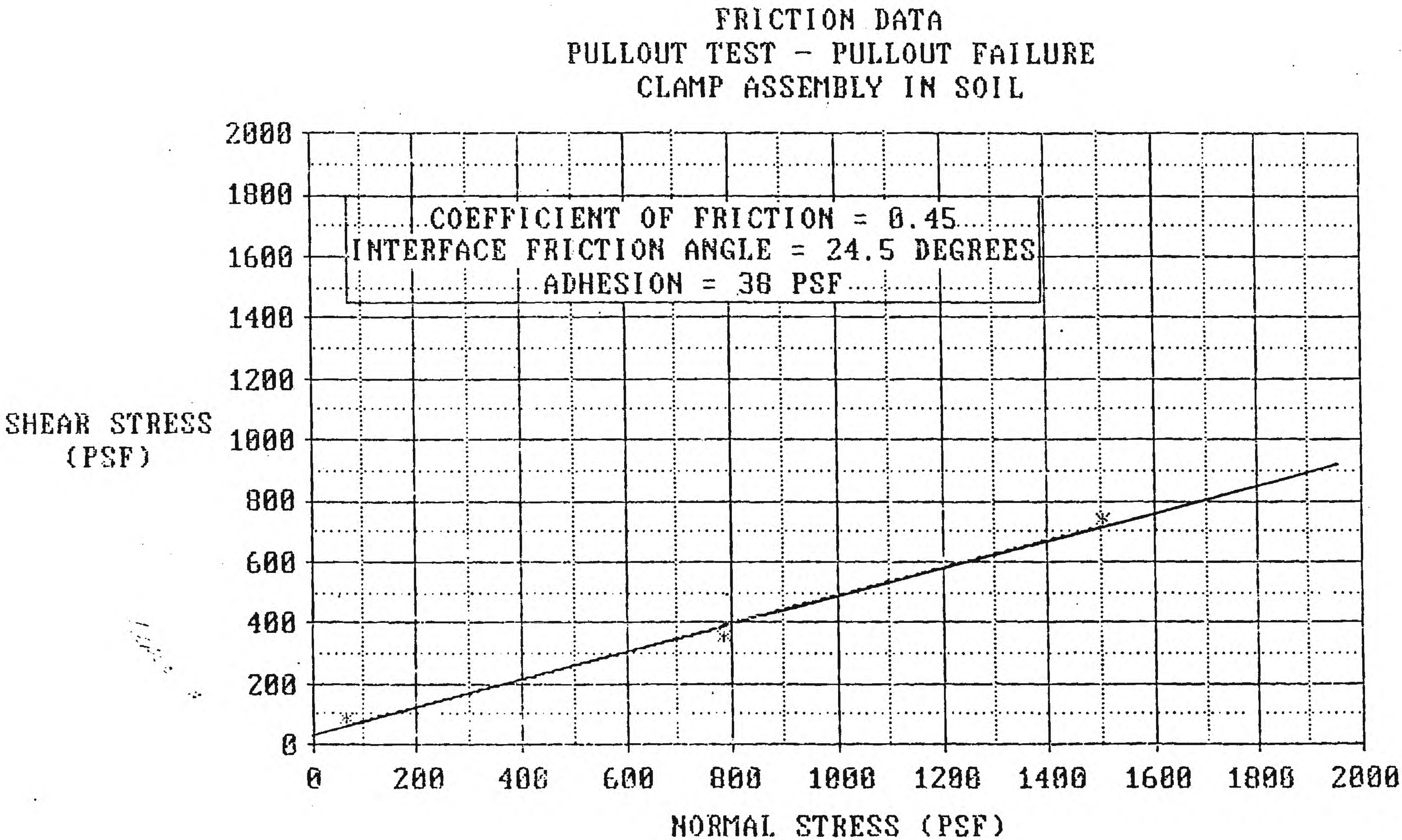


Figure 5-8. Friction Data for Clamp in Soil.

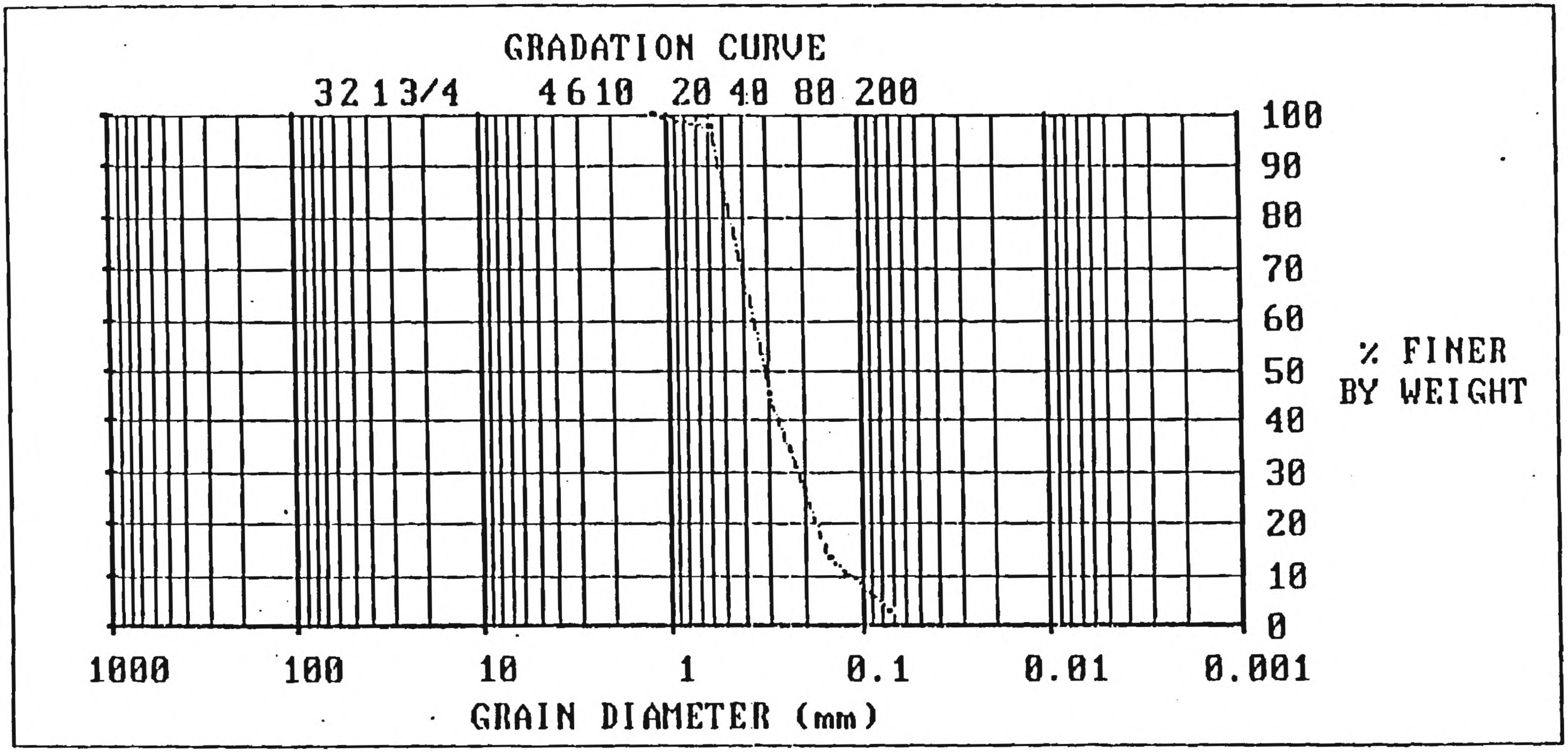


Figure 5-9. Gradation Curve for Concrete Sand (Cu = 3.1) (after Newman, 1986).

TABLE 5-1

CONCRETE SAND

RELATIVE DENSITY DETERMINATION

Mold Dimensions:

Diameter (in):	6.022
Depth (in):	6.138
Vol (ft^3):	0.101

e (max) Determination:

Wt. of Mold (g)	Wt. of Soil & Mold (g)	Wt. of Soil (g)	Void Ratio (e)
3518	7178	3660	1.08
3518	7158	3640	1.09
3518	7167	3649	1.08

Minimum Dry Density = 79.3 pcf

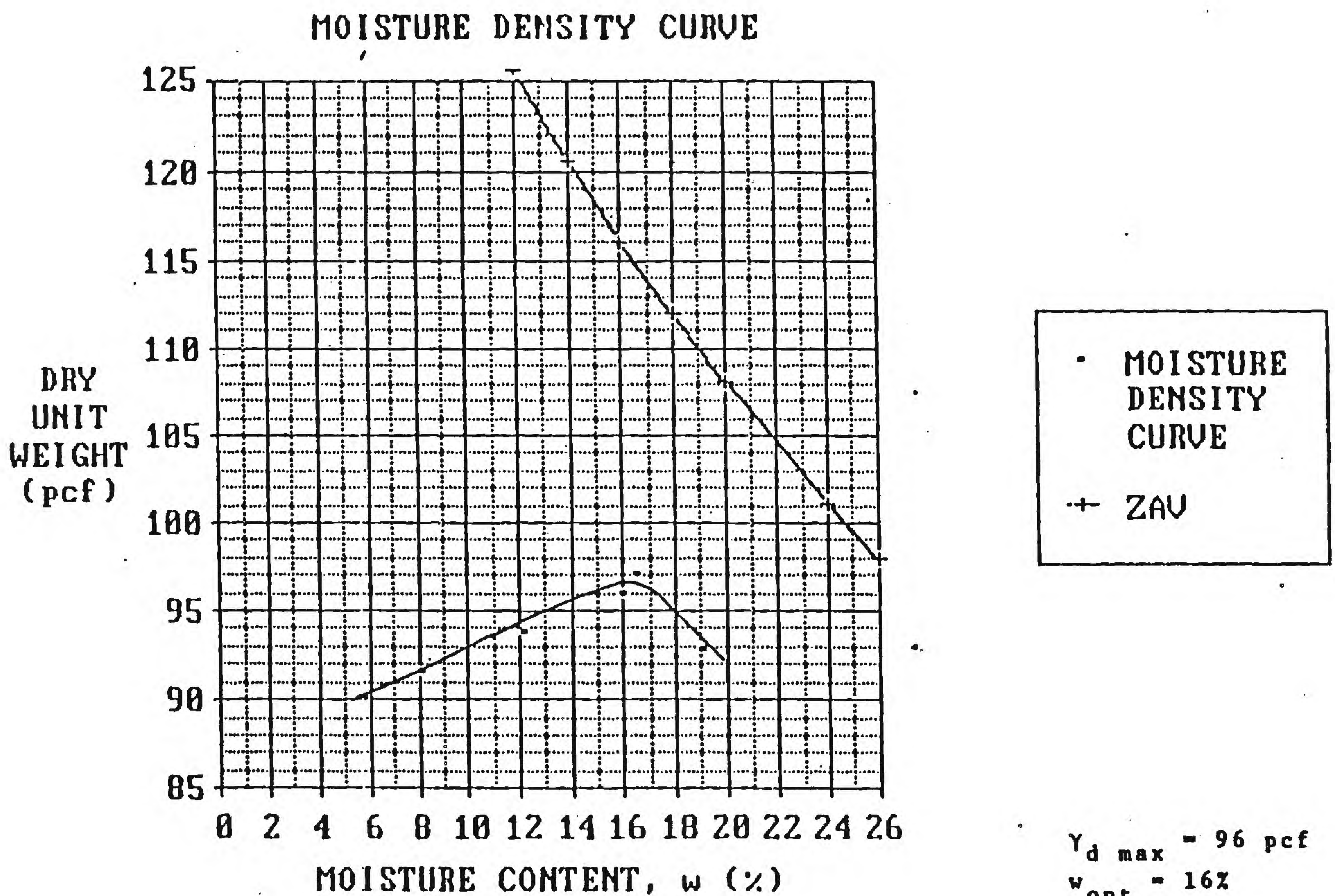
e (min) Determination:

Wt. of Soil (g)	Deflection (in)	Corrected Volume (ft^3)	Void Ratio (e)
3660	1.244	0.0807	0.655
3640	1.279	0.0801	0.652
3649	1.238	0.0808	0.662

Maximum Dry Density = 100.0 pcf

Relative Density:

Dr = 84%



$\gamma_{d\ max}$ = 96 pcf

w_{opt} = 16%

Figure 5-10 Moisture Density Curve for Concrete Sand (after Newman, 1986).

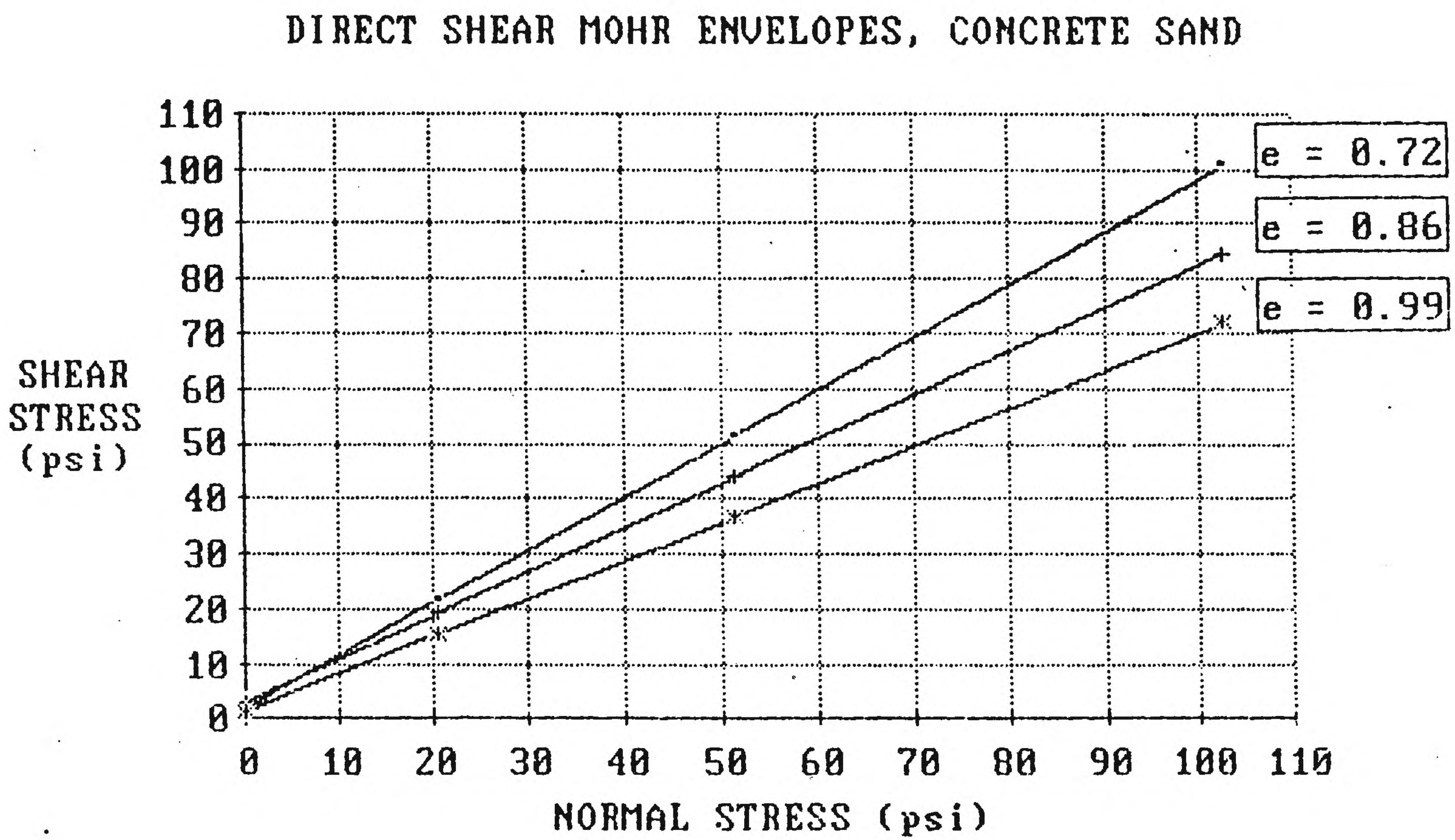


FIGURE 5-11 FAILURE ENVELOPES FOR CONCRETE SAND AT VARIOUS VOID RATIOS
(after Newman, 1986)

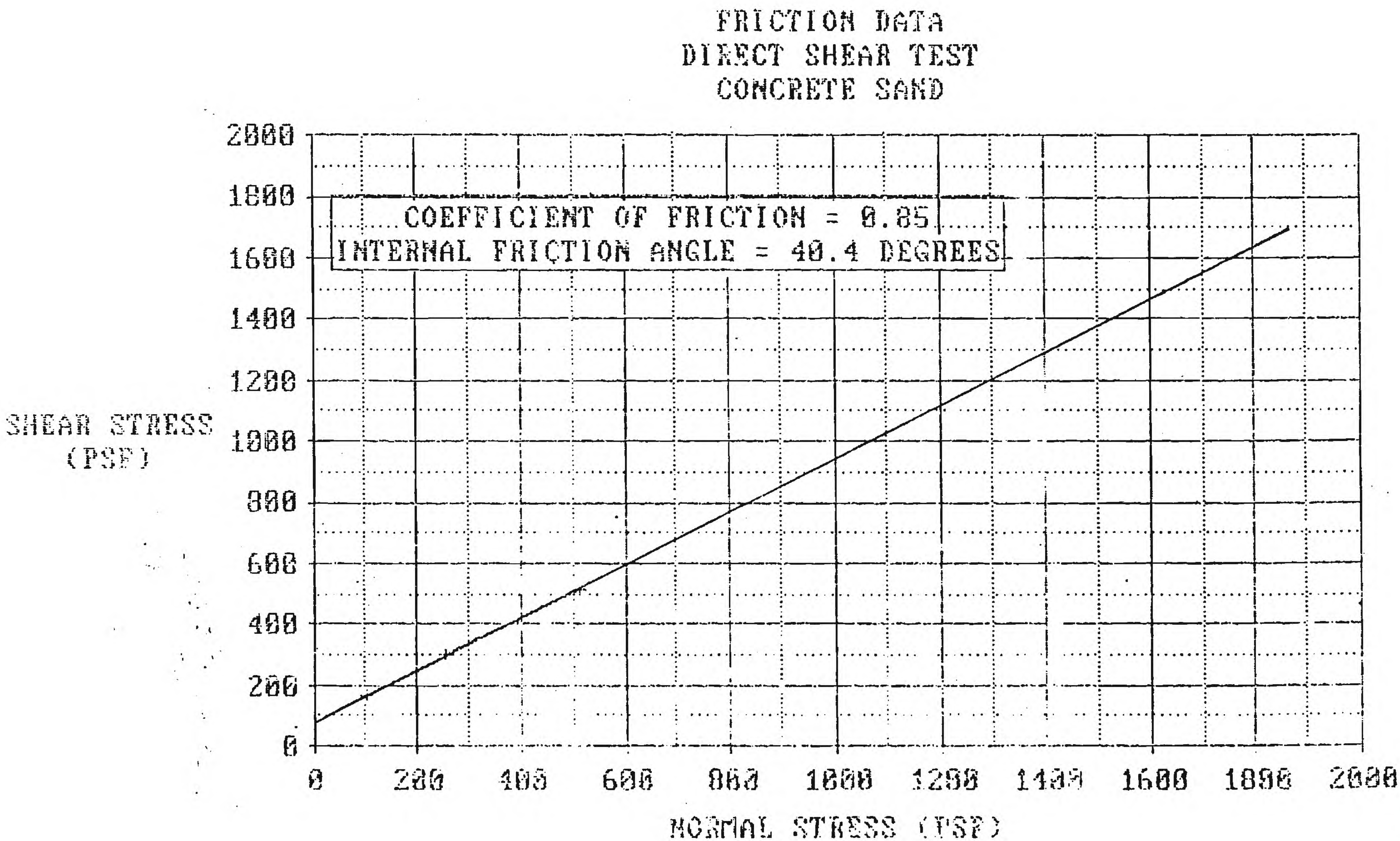


Figure 5-12 Friction Data for Concrete Sand.

(after Newman, 1986)

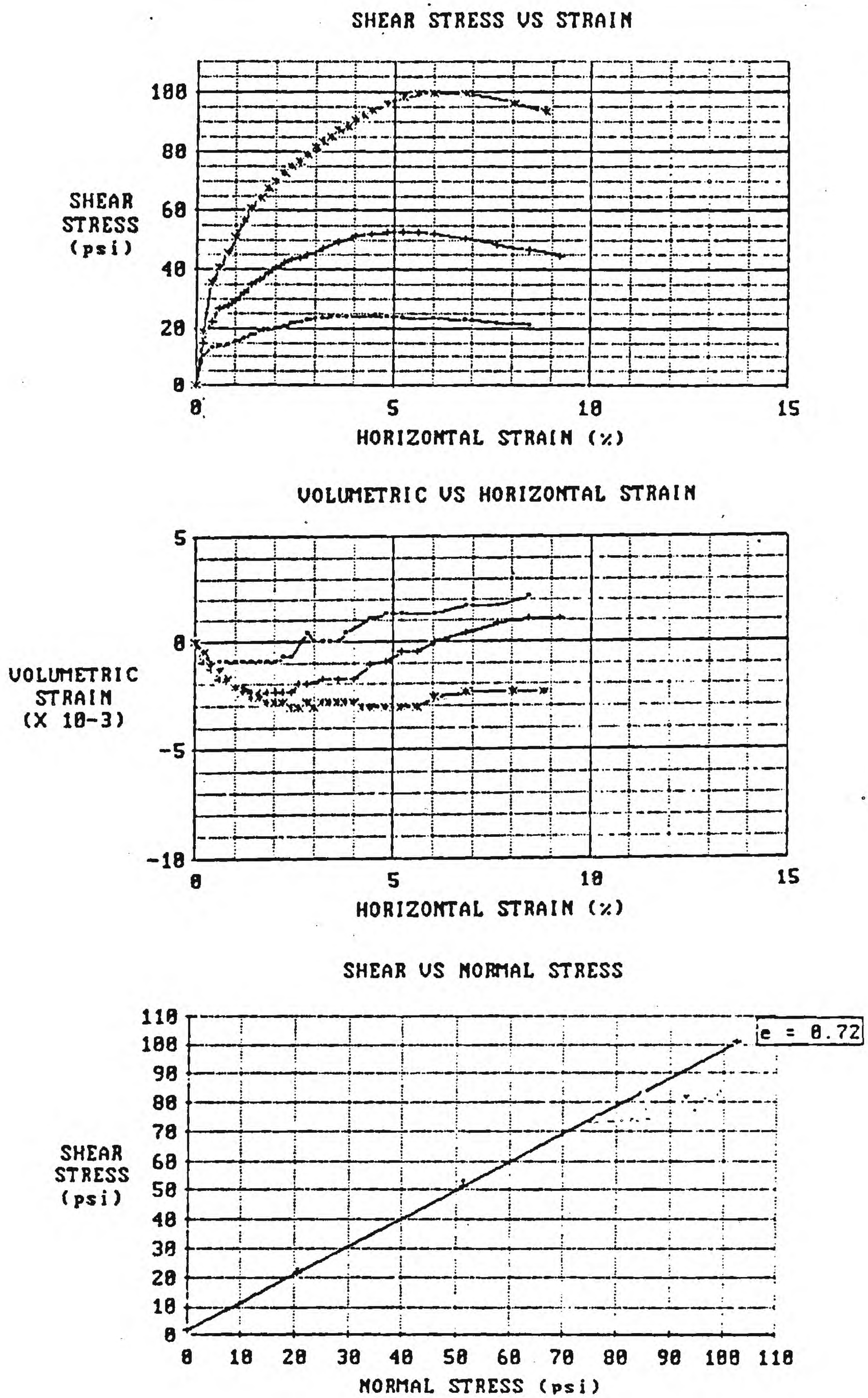
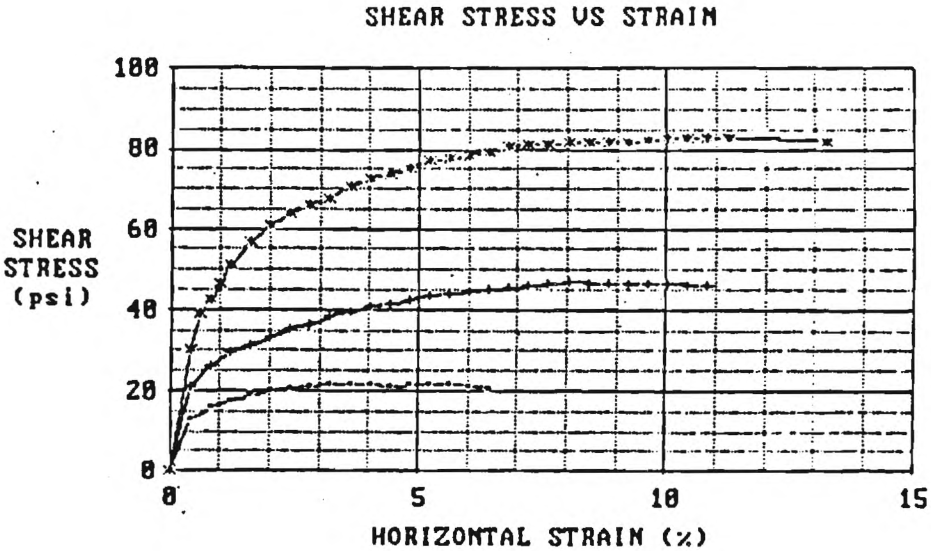
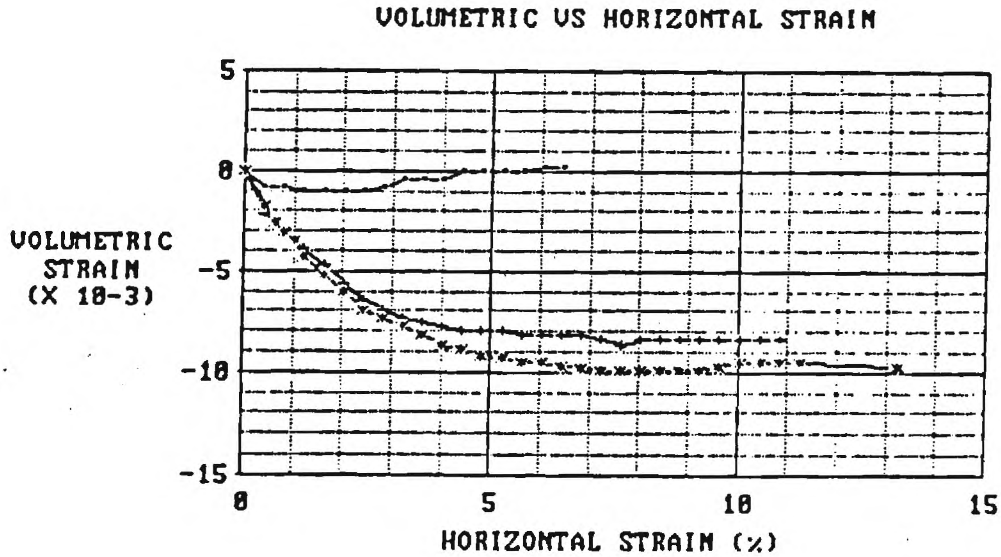


FIGURE 5-13. DIRECT SHEAR RESULTS FOR DENSE SAND (e = 0.72)

(after Newman, 1986)





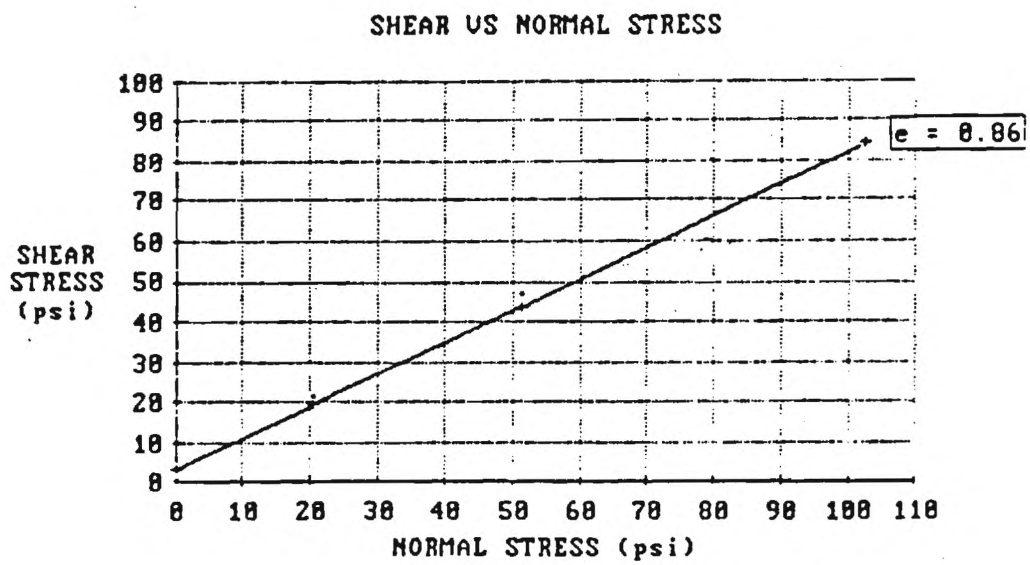
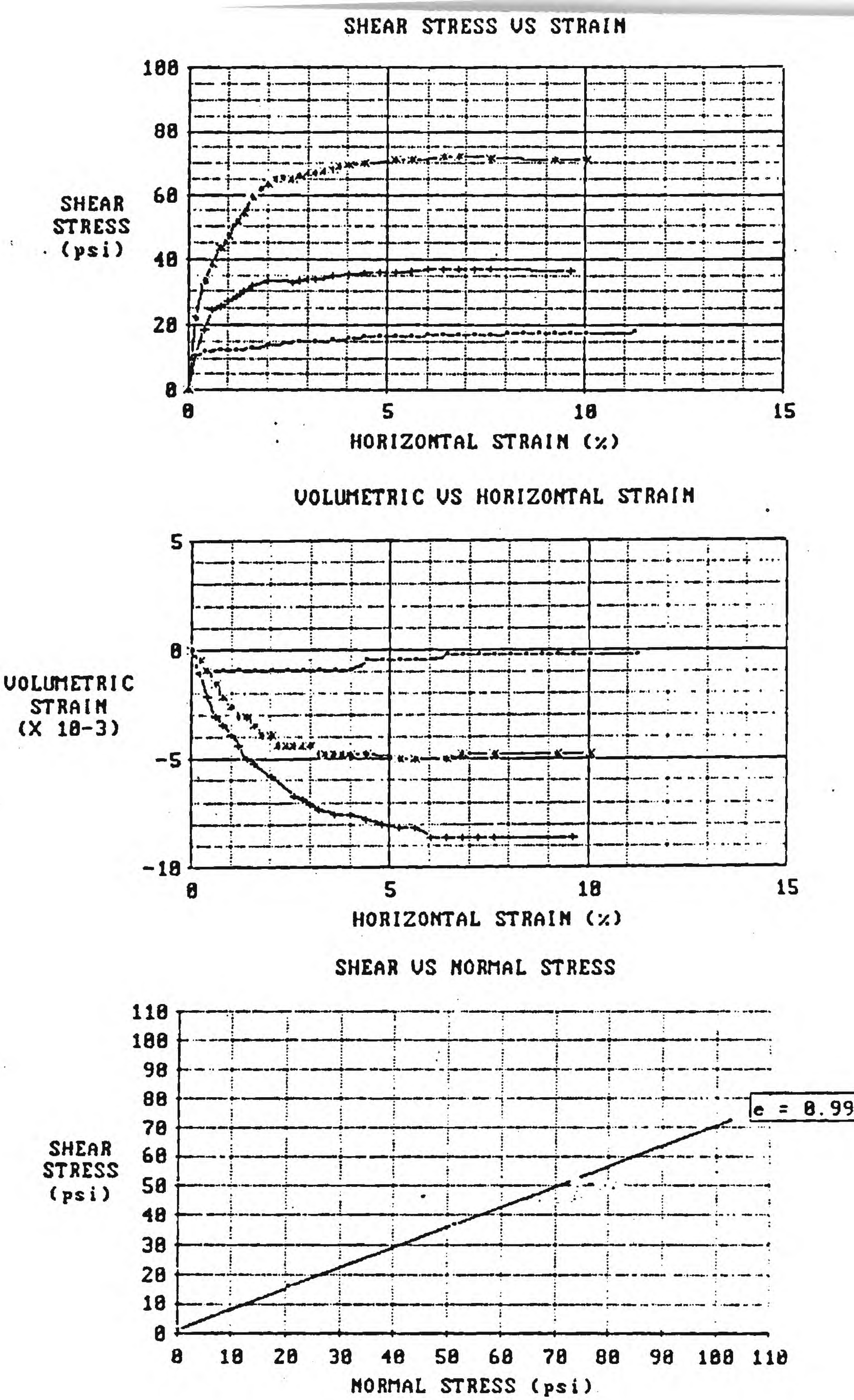


FIGURE 5-14 DIRECT SHEAR TEST RESULTS FOR MEDIUM CONCRETE SAND (e = 0.86) (after Newman, 1986)



67

FIGURE 5-15 DIRECT SHEAR RESULTS FOR LOOSE SAND (e = 0.99)

(after Newman, 1986)

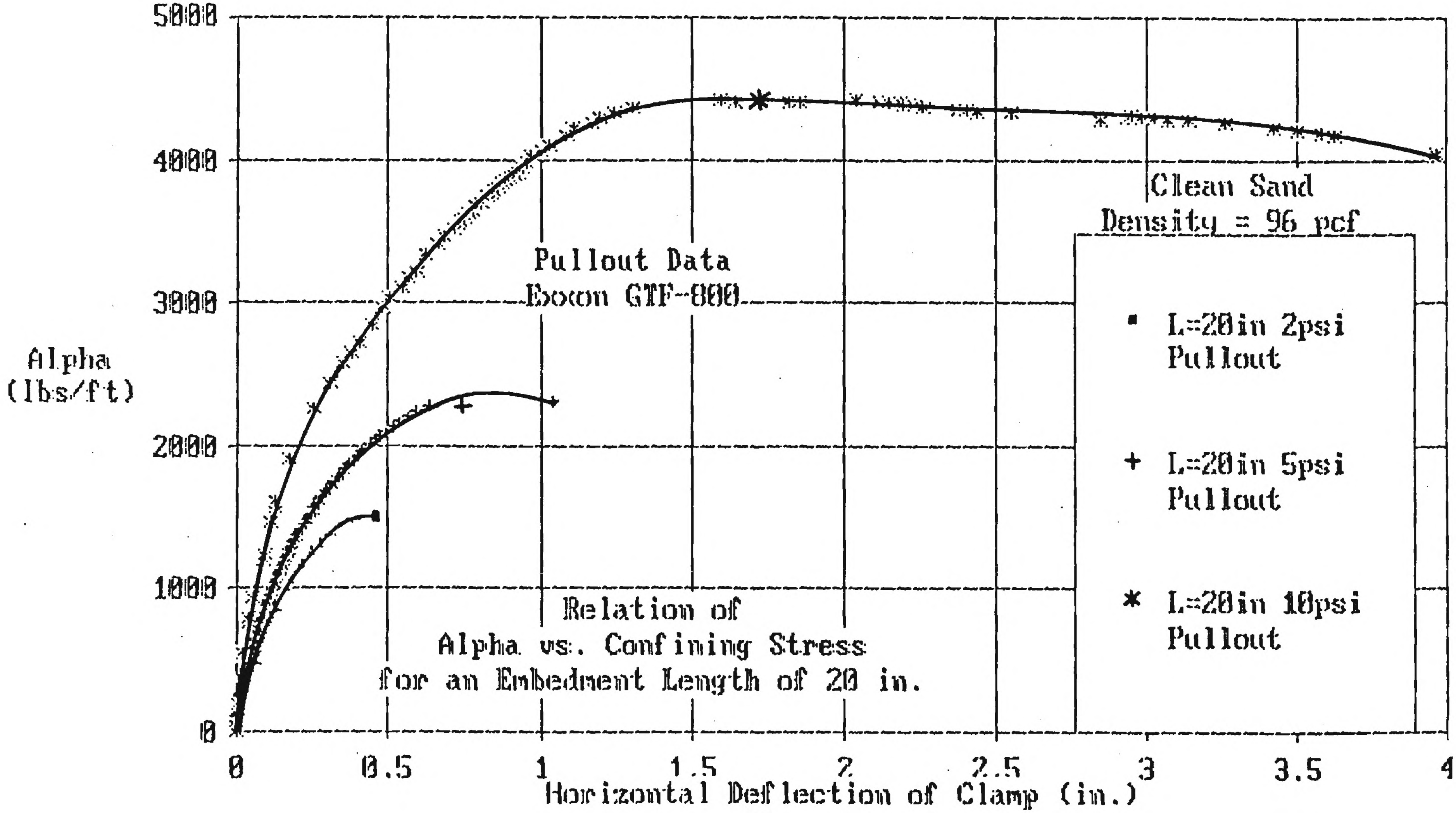
TABLE 5-2

RESULTS OF PULLOUT ANALYSES

Test #	Embedment Length (in)	Confining Stress (psf)	Alpha (lbs/ft)	Clamp Displacement (in)	Dry Density (pcf)	Degree of Compaction (%)	Failure Mode
1	40.0	1504	7008	3.25	82.1	86	Rupture
2	25.0	1504	5104	2.20	88.1	92	Pullout
3	25.0	1504	4881	3.25	88.4	92	PrematureRupt
4	20.0	1504	4427	1.60	87.1	91	Pullout
5	15.0	1504	3370	1.25	88.4	92	Pullout
6	20.0	784	2320	0.75	89.2	93	Pullout
7	20.0	352	1496	0.50	88.1	92	Pullout

Notes:

1. Confining stress equals weight of soil above geosynthetic layer plus bladder pressure.
2. Values of clamp displacement indicate the deflection at which rapid movement began; indicating begining of pullout or rupture.
3. Pullout indicates geotextile extraction from soil.
4. Rupture indicates tension failure of the geotextile.
5. Premature rupture of test number 3 was due to misalignment of clamping assembly.



5000
4000
3000
2000
1000
0
Alpha
(lbs/ft)
0
0.5
1
1.5
2
2.5
3
3.5
4
Horizontal Deflection of Clamp (in.)
Pullout Data
Exxon GTF-800
Clean Sand
Density = 96 pcf
L=20in 2psi
Pullout
L=20in 5psi
Pullout
L=20in 10psi
Pullout
Relation of
Alpha vs. Confining Stress
for an Embedment Length of 20 in.

Figure (5-16)

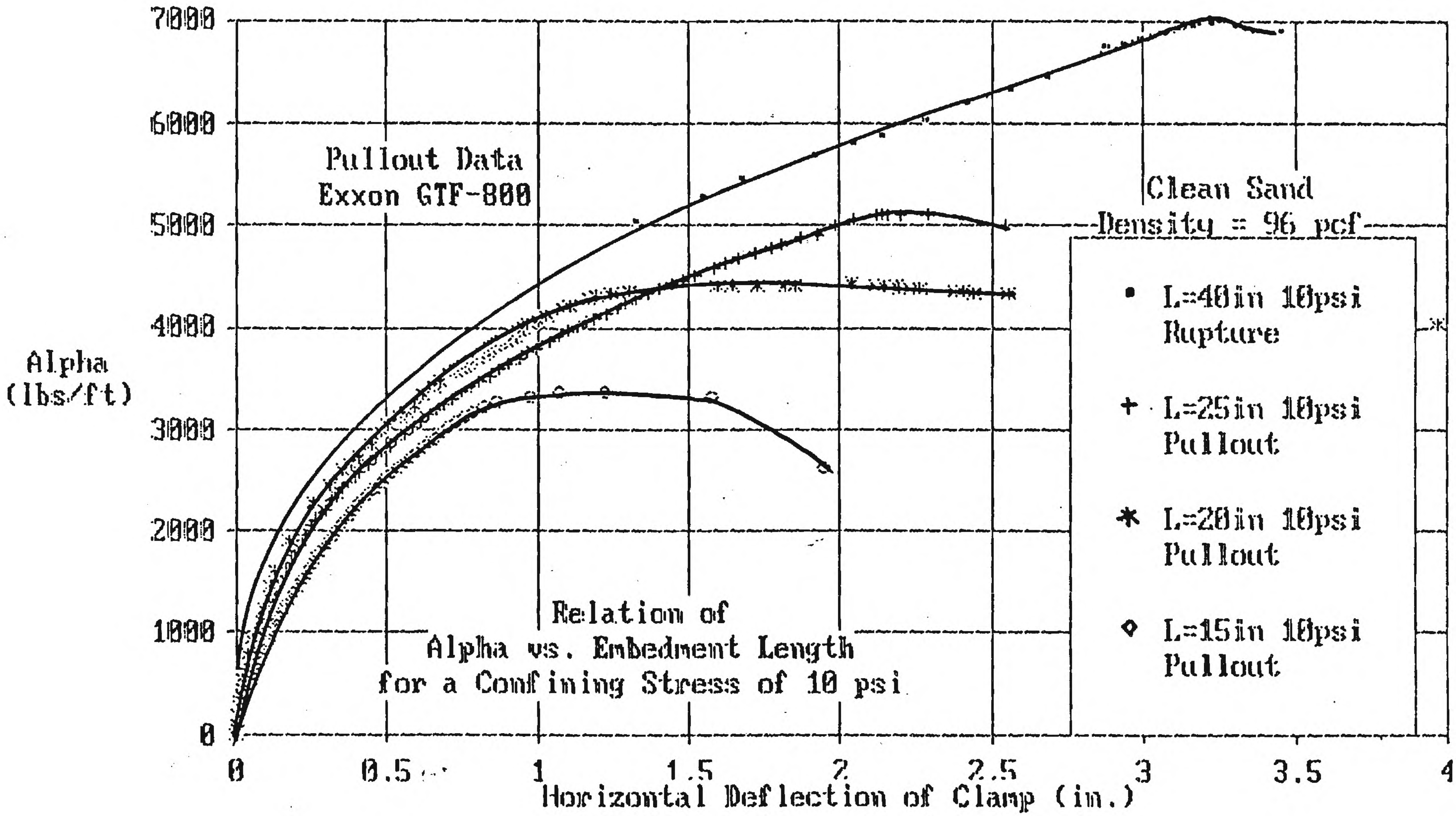
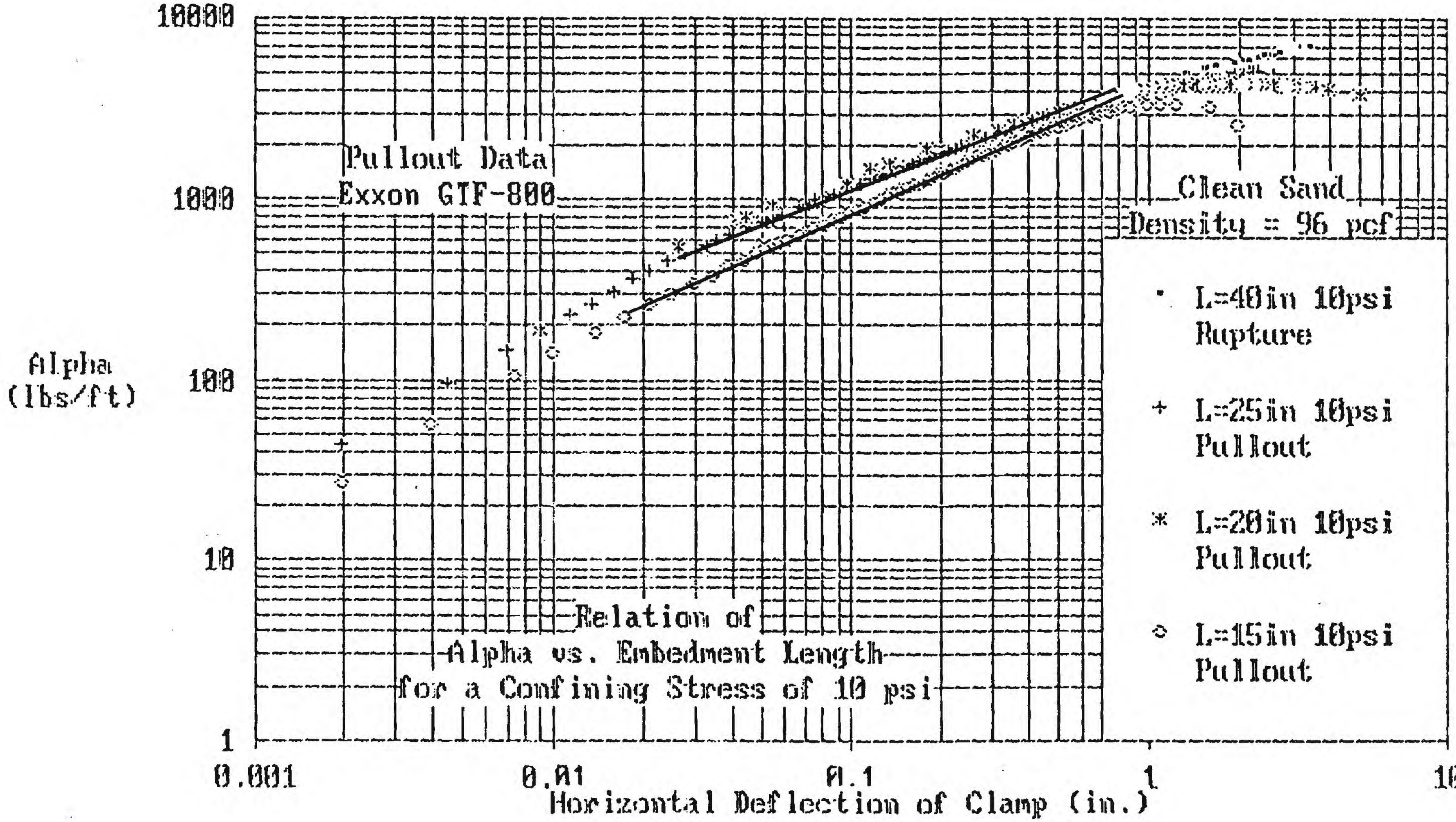

Pullout Data
Exxon GTF-800
Clean Sand
Density = 96 pcf
L=40in 10psi Rupture
L=25in 10psi Pullout
L=20in 10psi Pullout
L=15in 10psi Pullout
Relation of
Alpha vs. Embedment Length
for a Confining Stress of 10 psi
Alpha (lbs/ft)
7000
6000
5000
4000
3000
2000
1000
0
0
0.5
1
1.5
2
2.5
3
3.5
4
Horizontal Deflection of Clamp (in.)

Figure (5-17)



10000
1000
100
10
1
Alpha
(lbs/ft)
0.001
0.01
0.1
1
10
Horizontal Deflection of Clamp (in.)
Pullout Data
Exxon GTF-800
Clean Sand
Density = 96 pcf
L=40in 10psi
Rupture
L=25in 10psi
Pullout
L=20in 10psi
Pullout
L=15in 10psi
Pullout
Relation of
Alpha vs. Embedment Length
for a Confining Stress of 10 psi

Figure (5-18)

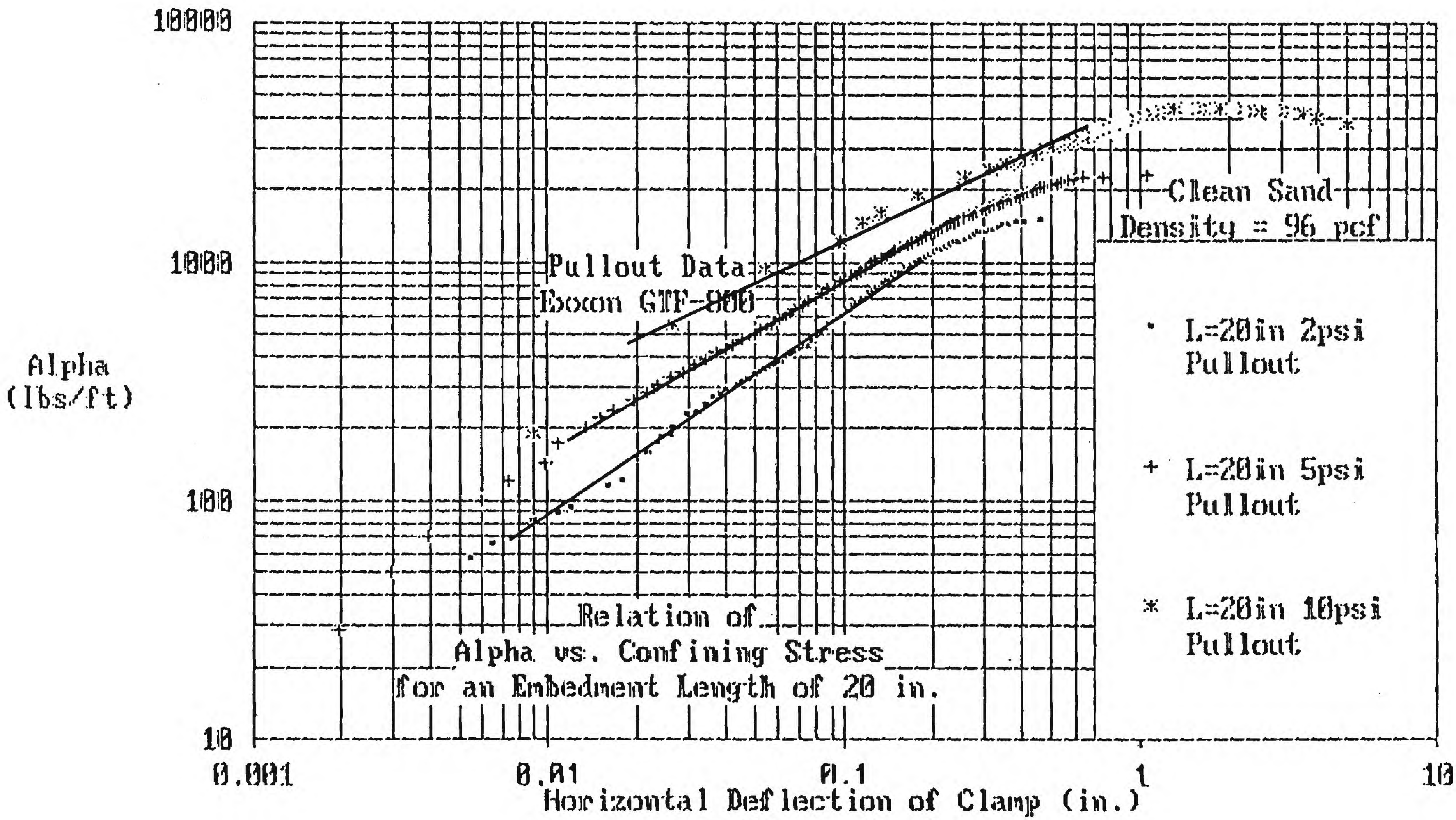
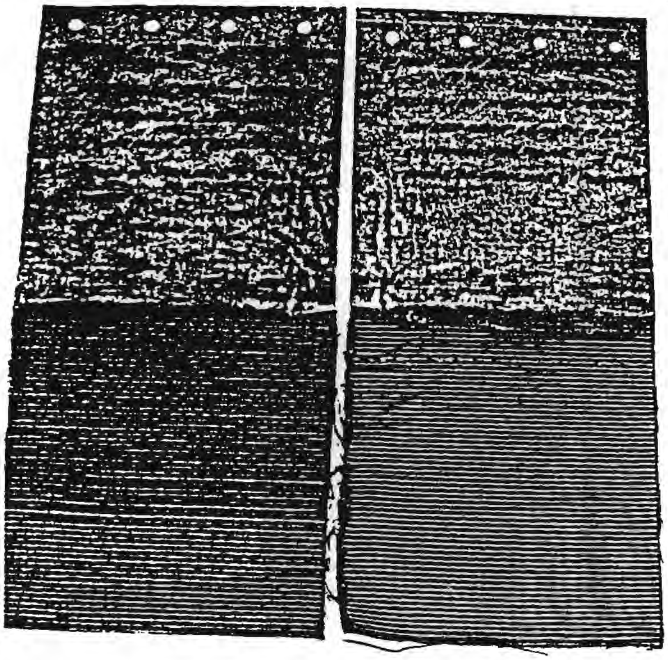
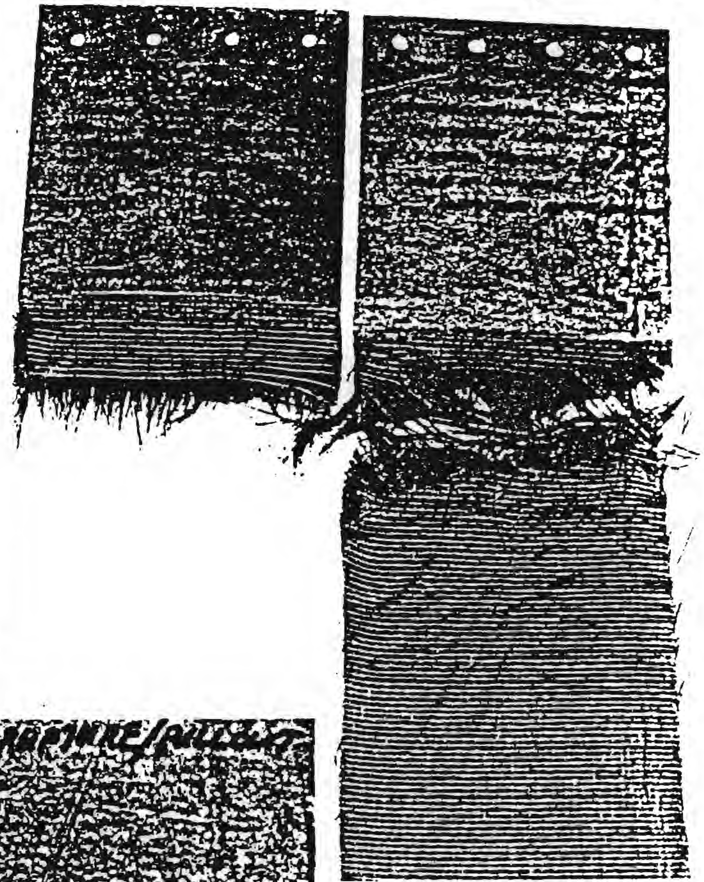

10000
1000
100
10
Alpha
(lbs/ft)
0.001
0.01
0.1
1
10
Horizontal Deflection of Clamp (in.)
Pullout Data
Exxon GTF-800
Clean Sand
Density = 96 pcf
L=20in 2psi
Pullout
L=20in 5psi
Pullout
L=20in 10psi
Pullout
Relation of
Alpha vs. Confining Stress
for an Embedment Length of 20 in.

Figure (5-19)



Pullout Failure



Rupture Failure

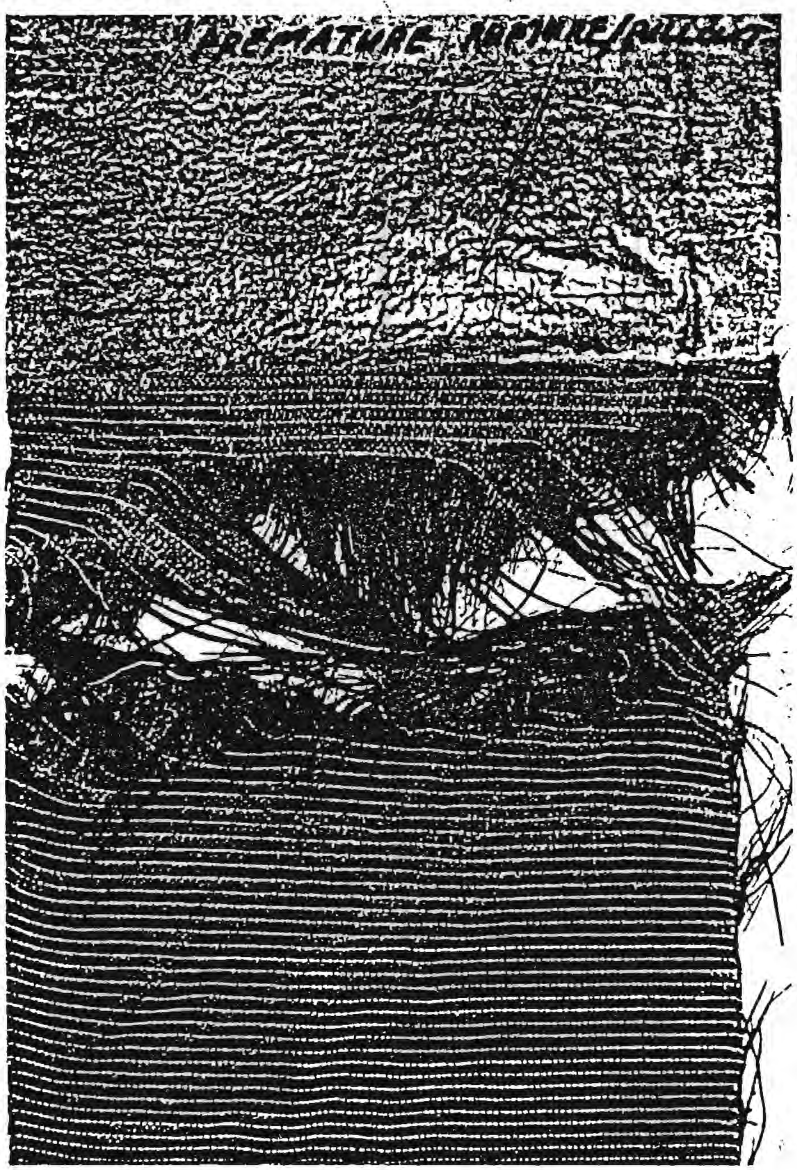


Figure 5-27P

Conclusions (relative to normal stress & embedment length)

For constant values of confining stress, the confined tensile strength increases as the embedment length is increased. This is true up to the point of rupture. It was found that rupture first occurs for embedment lengths between 25 and 40 inches for a confining stress of 10 psi. If the unconfined strength of the product is used, an embedment length of 32 inches can be calculated as being the length necessary for rupture to first occur at a confining pressure of 10 psi (Appendix E). This result is slightly conservative, but does agree with the limits observed.

Likewise, it was observed that if the embedment length is held constant at 10 inches, the confined pullout strength increases as the confining stress is increased for a range of 2 to 10 psi.

The unconfined tensile strength of the four layer composite (GTF 800) was found to be 6563 pounds/foot (Figure 6-3). In the confined analyses, for the rupture failure mode, the confined tensile strength was found to be 7000 pound/foot. Frictional forces at the interface account for the additional load carrying capacity of the geosynthetic.

Suggestions for Improvement (equipment)

The following improvements to the LSPCD are suggested in order to provide better effeciency and more accurate data. As with index, control and performance tests, the most important aspect is the consistency and repeatabilty of testing between laboratories. These suggestions for improvement are founded with this in mind.

1. Presently, the LSPCD is manually stress controlled. The addition of an in-line pressure sensing flow regulator would allow the device to be both

stress and deformation controlled.

2. Presently, the hydraulic cylinders of the LSPCD are connected to the pressure system by the rear ports of the cylinders only. This requires that the pressure hoses be removed and connected to the front ports in order to retract the cylinders. The addition of front and rear hoses connected in series with a hydraulic manifold and the pressure system would facilitate the operation of the LSPCD in a more efficient manner.

3. The ability to establish the distribution of stress and strain along the full length of a specimen would be of tremendous benefit.

CHAPTER 6

WIDE WIDTH TENSILE STRENGTH TESTING PROGRAM

Scope

The purpose of the wide width testing program for this study is two-fold. First, to establish the unconfined tensile strength of the GTF 800 stitch-bonded geotextile. Secondly, using the unconfined tensile strength, develop an unconfined tension creep program.

For this study, wide width tests were performed in general accordance with ASTM D4595. This procedure specifies that a sample width of 8.0 inches and gauge length of 4.0 inches be loaded at a constant deformation rate of 0.4 inches per minute. Roller grips fabricated for this study were used to clamp the geotextile and dial gages with 0.001 accuracy were used to measure strains.

Equipment Development (roller grip construction)

The equipment used in this testing program is relatively simple and consists of the following items: 20,000 pound tension/compression load frame, roller grips and dial gage. The roller grips were designed after existing technology. Figures (6-1) and (6-2P) are schematic drawings of the grips.

To measure deformations on the fabric, dial gages are attached directly to the fabric with plexiglass supports held by pin-cork stopper connections. This was observed to work extremely well for the geosynthetic being tested.

The advantage of roller grips over conventional wedge grip technology is that stress concentrations immediately adjacent to the grip are significantly reduced by the smooth, rounded surface. However, roller grips are not necessarily suited for some geosynthetics such as heavy gauge grids or drainage composites. Roller grips are typically used with woven and nonwoven geotextiles.

Methodology

As mentioned earlier, the wide width testing program followed ASTM D4595 procedures. The following step by step procedures are presented:

1. A sample of the geosynthetic is cut to 8.0 by 40.0 inches. Care should be taken to maintain clean, straight edges as this can cause premature ravelling of the edges during testing;
2. The two halves of the roller grip set are attached to the loading frame and adjusted for level and plumbness;
3. The lock bar of the upper grip is removed and wrapped in one end of the sample one revolution. The wrapped lock bar is then inserted into the triangular slot of the upper grip. Care should be taken to center the sample on the cylinder in order to avoid eccentric loading;
4. The grip cylinder is rotated in its yoke until the fabric wraps back on itself. At that point, the lock pins are placed in both ends of the grip cylinder locking it to the support yoke;
5. The grip cylinder of the lower grip is removed by taking out the two axis bolts. Step 3 is repeated. The removal of the lower cylinder from its yoke allows the operator to mount the sample while maintaining proper alignment;
6. The sample is rolled up as in step 4 and axis bolts replaced. At this point, the system is adjusted for level and plumbness. Approximately 8

inches are needed between the two grip cylinders in order to install the dial gage supports;

7. At this point, the system must be pretensioned before installation of the dial gages can proceed. This is done by turning on the loading machine and applying a small load. For this study, a preload of approximately 25 pounds was used. The preload is necessary to provide a rigid surface for mounting dial gages and to establish a zero reference;

8. A four inch gauge length is then layed off in the center of the sample and the dial gages installed. The metal pins (finishing nail) of the gage supports are forced through the fabric at the exact limits of the 4 inch gauge length along the vertical centerline of the sample. Cork stoppers are then placed over the metal pins on the back side of the sample. The dial gages are then properly seated and a zero reading recorded;

9. At this point, the test can begin. The proper rate of deformation is set on the load machine and data sheets prepared. The machine is then turned on and readings obtained at the desired intervals. For this study, readings were obtained every 0.5 minutes at the begining and near the end of the test. Intermediate readings were taken every 1.0 minutes. For each reading, values of load, deflection and time are recorded.

The test is then continued until the geosynthetic fails in tension. Readings are taken up to and past failure if possible. This data yields an elasto-plastic stress-strain relation typical of polymeric materials.

Results

Results of the wide width tests are presented in Figure (6-3) as alpha (lbs/in) versus strain (%). A total of four tests were performed with an average unconfined tensile strength of 547 pounds per inch of width and an average secant modulus of 4783 lbs/in.

Wide width data taken from an article written by the manufacturer along with other index test results are presented in Tables (2-1),(2-2) and (2-3). The values of wide width strength as observed in this study are approximately seven percent lower than that reported by the manufacturer. This discrepancy can possibly be attributed to differences in roll properties between the two testing programs.

Specific index test results for the products used in this study are given in Appendix (D).

Suggestions for Improvement

For the geotextile tested, the roller grips were observed to function very well. However, the all-thread bolt used to connect the two halves of the roller grips to the load frame were found to bend slightly due to the 1.5 inch offset of the grip cylinder radius. This does not affect the test results since the sample aligns itself accordingly when placed in tension. The only problem is that with time the lock nuts become difficult to thread over the bent region. To alleviate this it is suggested that another degree of freedom be added to the system. This can be done by adding a swivel type joint or perhaps a ball joint between the roller grip support yoke and the loading frame.

In this study, dial gages mounted directly on the geosynthetic were used. This works very good up to the point at which the specimen begins to fail and gives good results for determining modulus. However, tracking strain from the peak load down through the breaking point is difficult due to the increase in deflection rate at that point. The use of LVDT's in the place of the dial gages would allow the post failure region to be monitored. The trade off in using LVDT's is set-up time. Depending on the nature of the testing program, one method may be chosen over another.

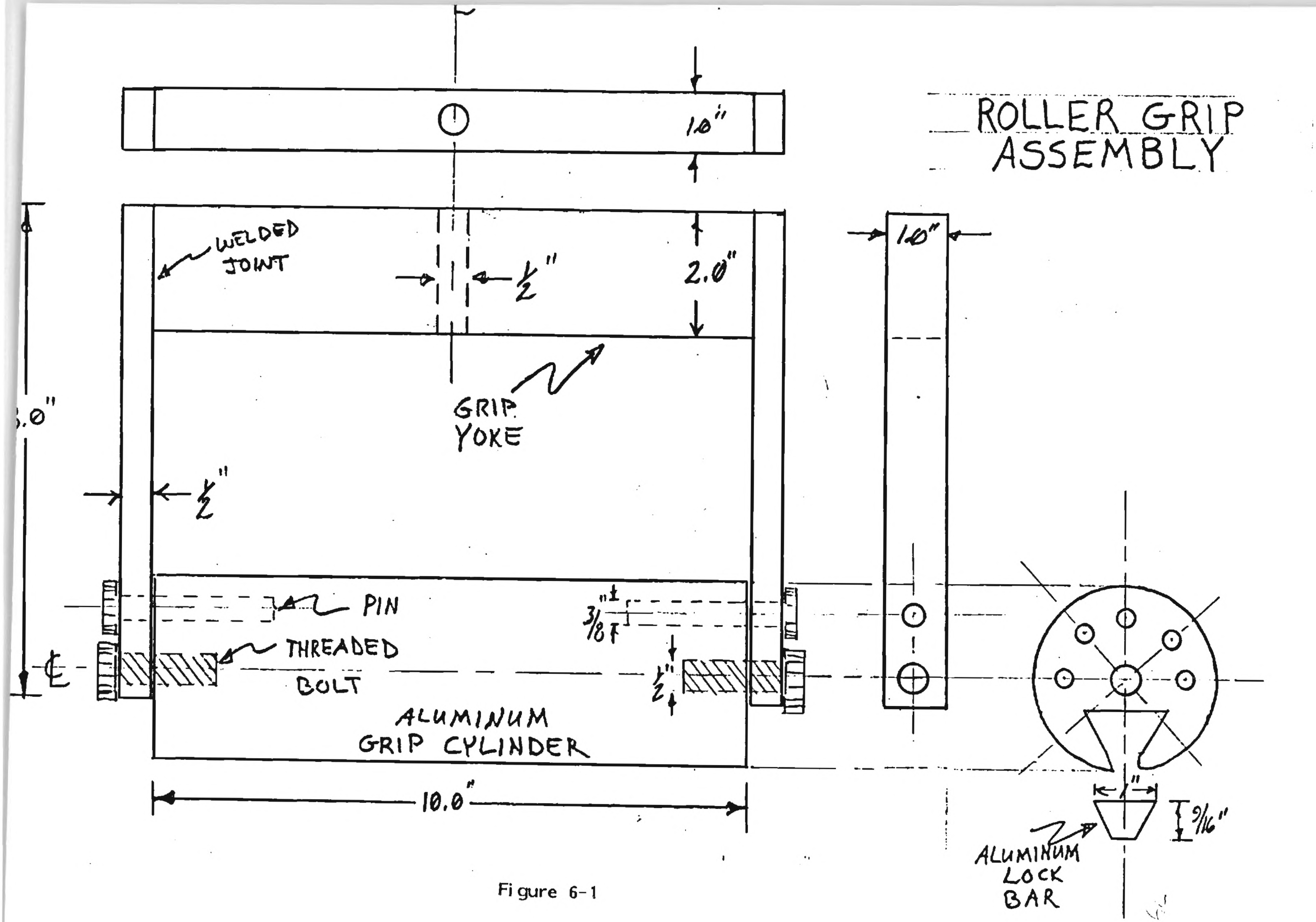

ROLLER GRIP ASSEMBLY
1.0"
WELDED JOINT
1/2"
2.0"
GRIP YOKE
1/2"
PIN
THREADED BOLT
3/8"
1/2"
ALUMINUM GRIP CYLINDER
10.0"
1.0"
1"
9/16"
ALUMINUM LOCK BAR

Figure 6-1

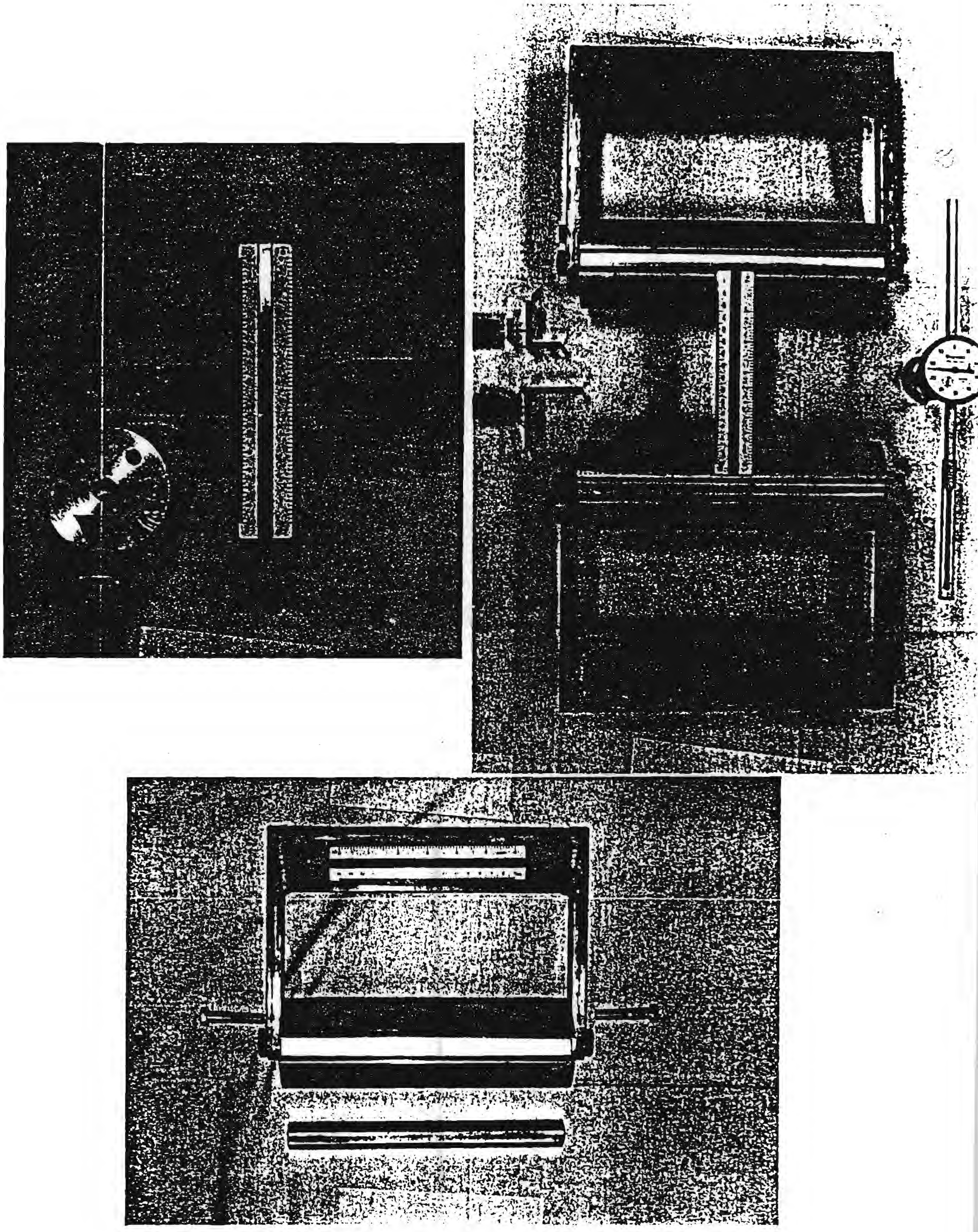


Figure 6-2P (Roller Grip Assembly)

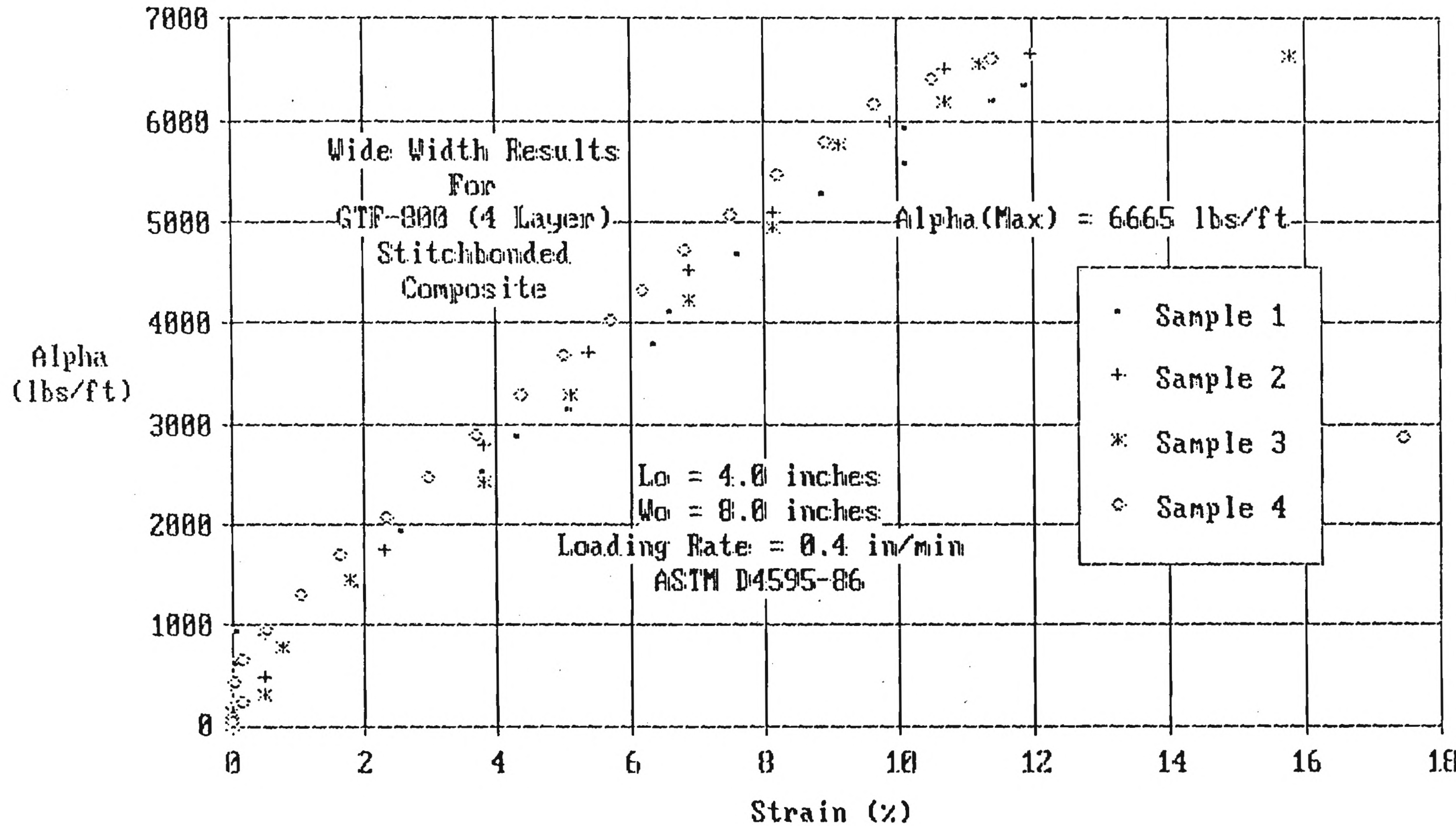

7000
6000
5000
4000
3000
2000
1000
0
Alpha
(lbs/ft)
Wide Width Results
For
GTF-800 (4 Layer)
Stitchbonded
Composite
Alpha(Max) = 6665 lbs/ft
Sample 1
Sample 2
Sample 3
Sample 4
Lo = 4.0 inches
Wo = 8.0 inches
Loading Rate = 0.4 in/min
ASTM D4595-86
0
2
4
6
8
10
12
14
16
18
Strain (%)

Figure (6-3)

CHAPTER 7

TENSION CREEP TESTING PROGRAM

Scope

The purpose of the unconfined tension creep program is to establish a relation that can be used to predict the time dependent deformations of geotextiles under sustained tensile loads.

For this study, creep tests were performed at stress levels, Dx, of 60, 50, and 30 percent of the product's average ultimate unconfined tensile strength as determined from the wide width testing program. A forty day test duration was determined to be adequate for the scope of this study.

Equipment Development

The equipment used for the tension creep program consists of roller grips with a loading frame fabricated for this study. Figures (7-1) through (7-4P) are schematic drawings and photographs of the load frame and its use. Dial gages are used to measure strains as in the wide width program.

The load frame uses a mechanical advantage to apply the desired load or stress. A stress is applied to the specimen by means of a load amplifying lever arm. The load frame built for this study was constructed with a 10:1 mechanical advantage. Calibration curves for the frame's two specimen cells are presented in Appendix (E).

Construction of the load frame consists of arc-welded 2 in X 2 in X 3/16 in structural steel tubing. The entire frame is supported by a table built of 2 in X 2 in X 1/8 in steel angle. Roller grips are attached to the load frame in the same manner as the wide width program. Loads are applied by dead weights suspended at the end of the lever arm.

Methodology

Set up of the tension creep test is similar to that of the wide width program.

1. Steps 1 through 6 of wide width program are followed exactly;

2. The sample is pretensioned by applying the load induced by the weight of the lever arm and empty weight hanger.

3. The lever arm is brought up to its upper limit by adjusting travel nuts on the upper and lower grips. This serves as the initial starting reference point;

4. Step 8 of the wide width program is followed exactly.

5. At this point, record keeping should be initiated and start-up procedure discussed. Since creep results are normally expressed in log time, readings are taken at doubling intervals (i.e. 6 sec., 12 sec. 30 sec., 1 min, 2 min., etc.) After 24 hours, readings need only be taken on a daily basis. Laboratory temperature should be kept at approximately 70 degrees F;

6. To start the test, a 4-inch C-clamp is used to support the lever arm in its upper position. The desired weight is placed in the weight hanger. The stress felt by the specimen includes the contribution of the lever arm and weight hanger. A stopwatch is started at the same instant the C-clamp is removed; this initiates the test sequence; and,

7. At the conclusion of a test, the weights are removed from the hanger, the C-clamp replaced to support the lever arm and the specimen removed.

Results

Tension creep results are commonly expressed using a series of graphs relating the stress level, Dx, the total strain, strain rate, and the elapsed time. These analyses may be modeled using the Mitchell-Singh creep model,(1968).

The Mitchell-Singh creep model, developed for soils, is a three parameter expodential phenomenological relationship developed from emperical curve-fitting techniques that does not necessarily imply anything about the mechanisms underlying the deformation processes associated with the creep of soils. However, the model has been found suitable for the description of the creep rate behavior of a wide variety of soils (18). Likewise, the model has been found suitable for a variety of geosynthetics.

For this study, the Mitchell-Singh model was employed to estimate the time to failure as a function of stress level. The governing equation in this case is:

$$Tf = \left[\frac{(Ef - Ei)(1 - m)}{Ae^{(aDx)}} + 1 \right]^{\left(\frac{1}{1 - m}\right)} \tag{7-1}$$

where	Tf	– time to failure	(T)
	Ef	– strain at failure for known stress level	(–)
	Ei	– strain at unit time	(T)
	m	– slope of LogStrainRate vs. LogTime plot	(–)
	a	– slope of LogStrainRate vs. StressLevel plot	(–)
	A	– intercept of LogStrainRate vs. StressLevel	(–)
	Dx	– stress level	(–)

Figures (7-5) through (7-8) show the four relationships necessary for the Mitchell-Singh model. The creep phenomena for the GTF-800 product appears to be typical in all respects. Namely, as the stress level increases, the time to failure decreases and the strain rate increases. In addition, for stress levels less than a product specific value, failure does not occur over the time intervals tested.

At a stress level of 60%, rupture occured after 15 days. At a stress level of 50%, failure did not not occur during the 40 day test interval. However, from Figure (7-6) the increase in strain rate near the end of the test suggests that failure was imminent. At a stress level of 30%, the strain rate decreases log-linearly as a function of log-time.

The Mitchell-Singh model was employed to estimate the time to failure as a function of stress level. However, it was found that for the product investigated (GTF-800) under the range of stress levels used, the Mitchell-Singh model did not accurately predict the time dependent deformations. The model over predicted the rate of strain at large values of time and therefore over predicted the total rate of strain.

Suggestions for Improvement

The following modifications are suggested for improvement of the tension creep system:

1. The lever arm support should be increased to facilitate the testing of high elongation geosynthetics.

2. Presently, the lever arm and upper grip assembly are mounted by 1/2 inch hardened steel bolts with no bushing or bearing. The calibration of the lever arm ratio revealed that the friction in the system resulted in a residual stress of 1.0 pound when unloaded. However, the use of roller bearings at the two points of rotation is suggested to reduce system friction.

3. It is strongly advised that the entire unit be bolted or in some way rigidly attached to the laboratory floor.

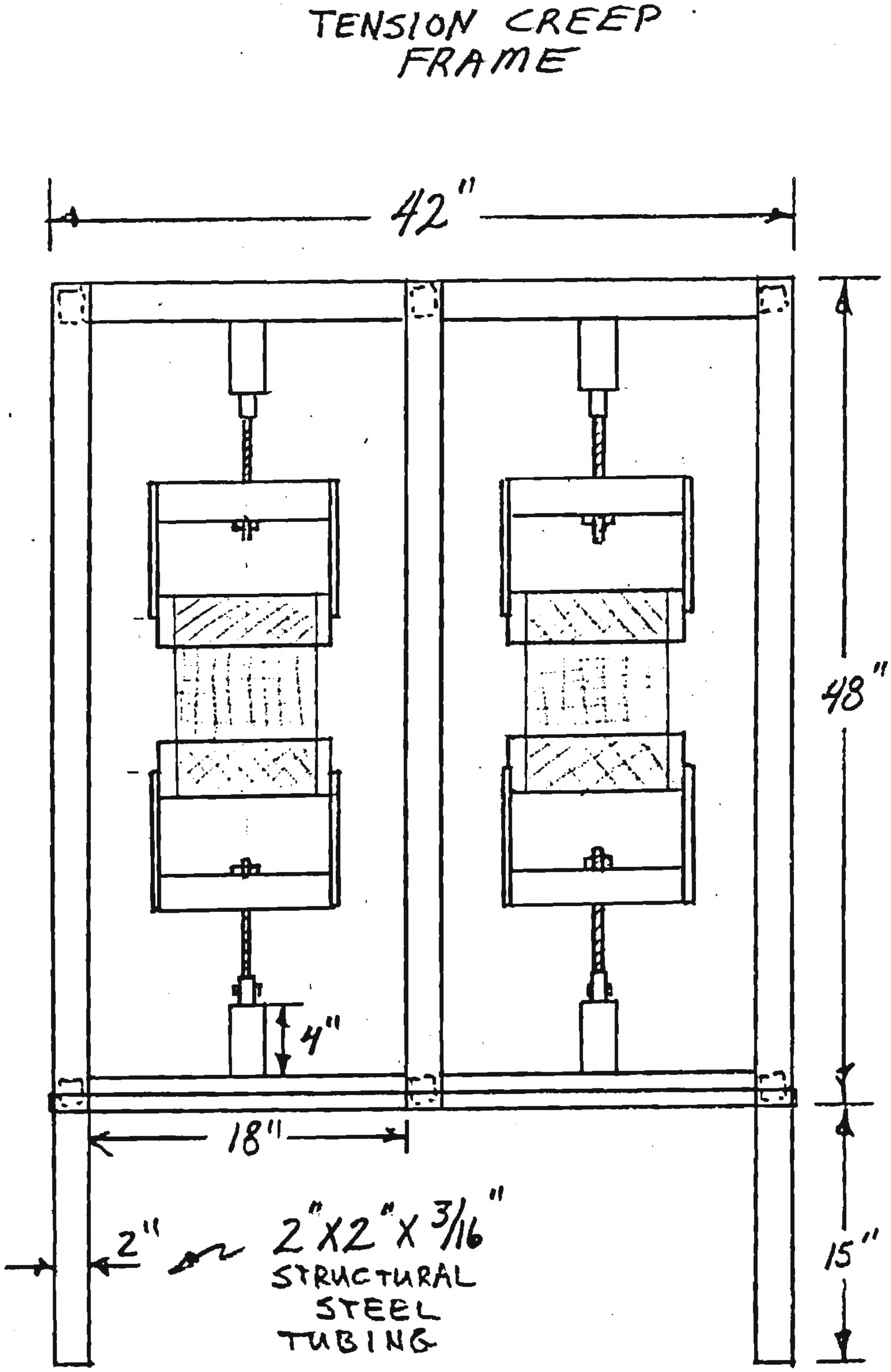


Figure 7-1 (Tension Creep Frame)

TENSION CREEP FRAME

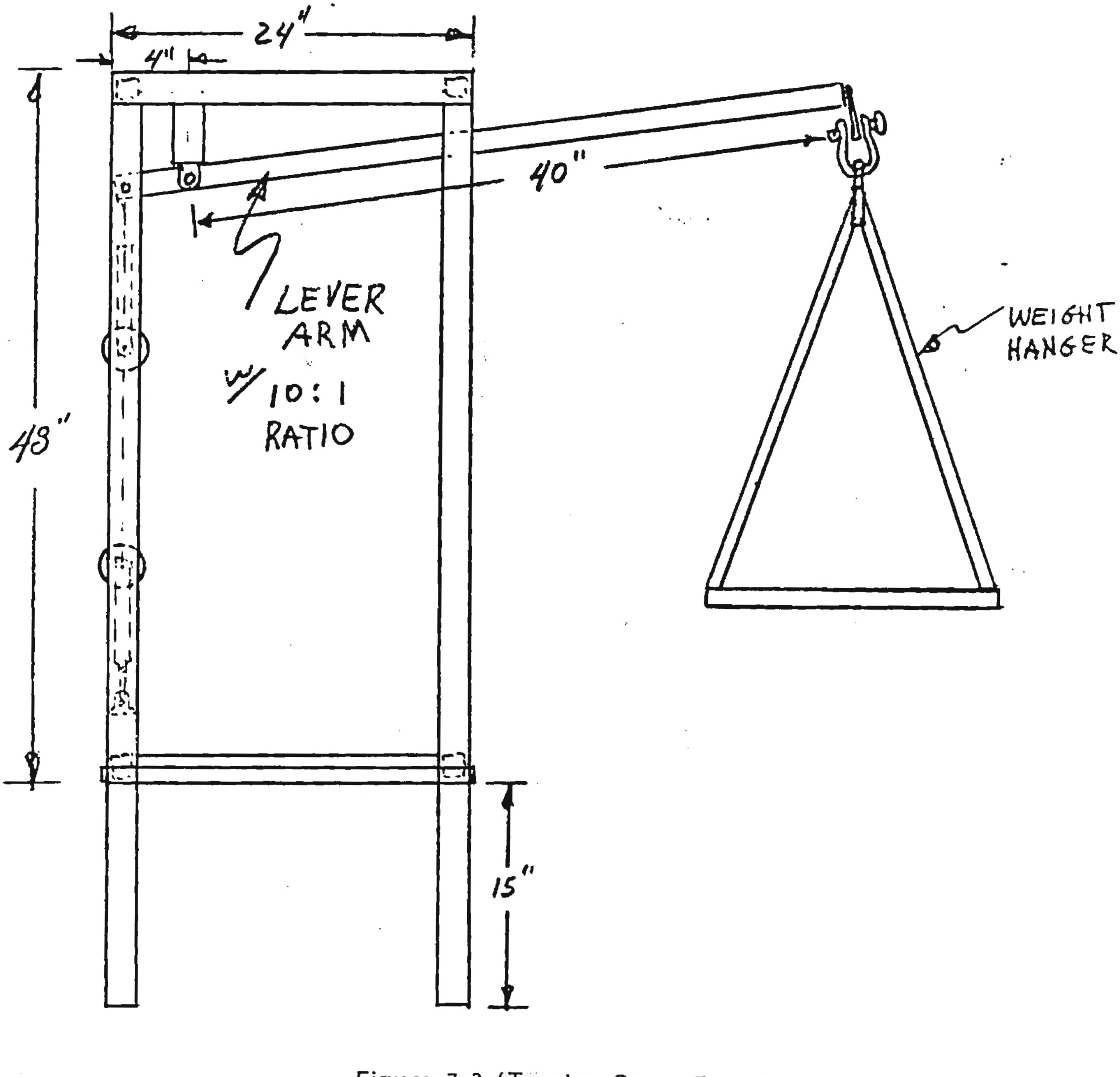


Figure 7-2 (Tension Creep Frame)

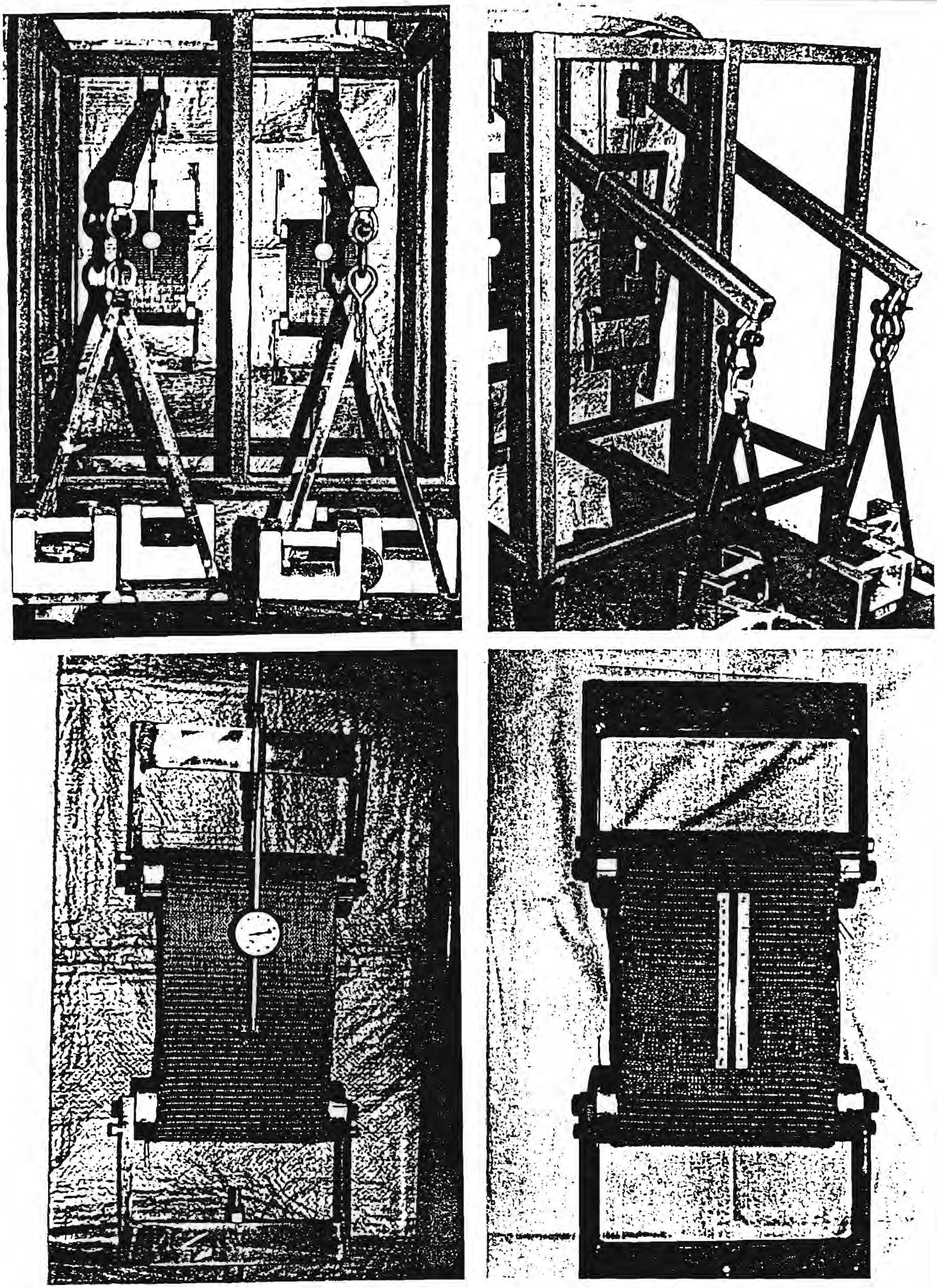


Figure 7-3P (Tension Creep Apparatus)

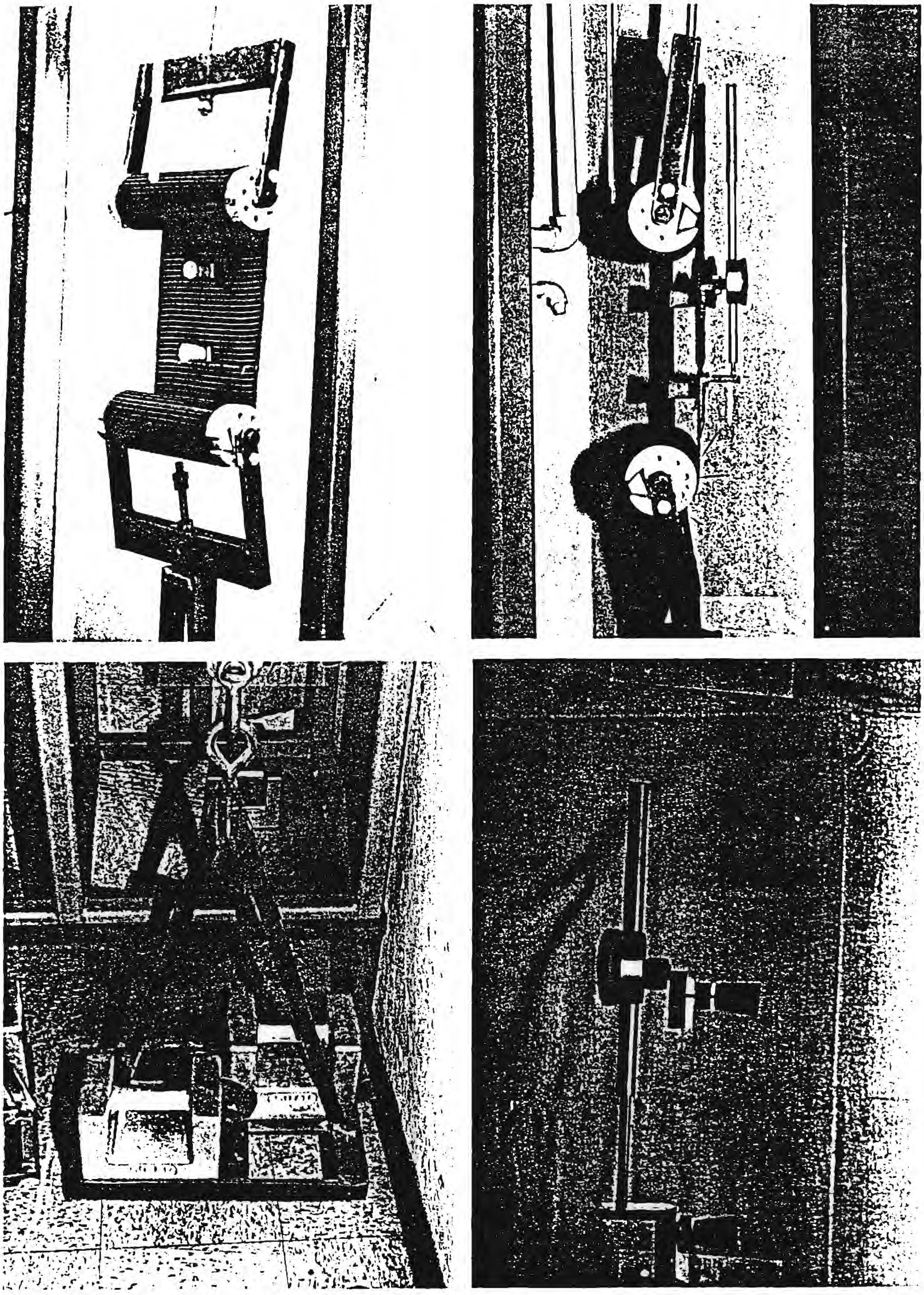


Figure 7-4P (Roller Grip & Strain Indicator)

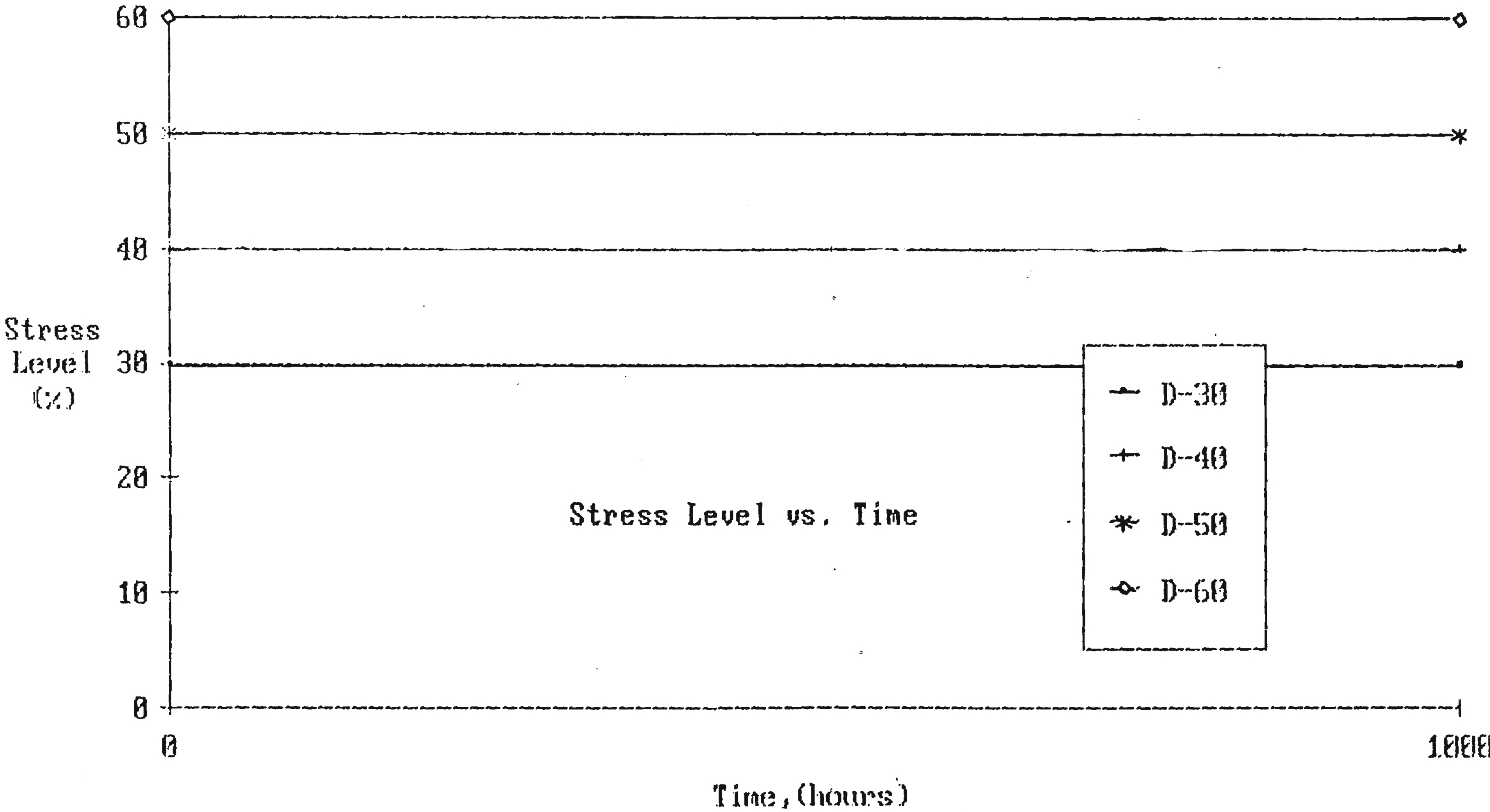

60
50
40
30
20
10
0
Stress
Level
(%)
Stress Level vs. Time
D--30
D--40
D--50
D--60
0
1000
Time,(hours)

Figure (7-5)

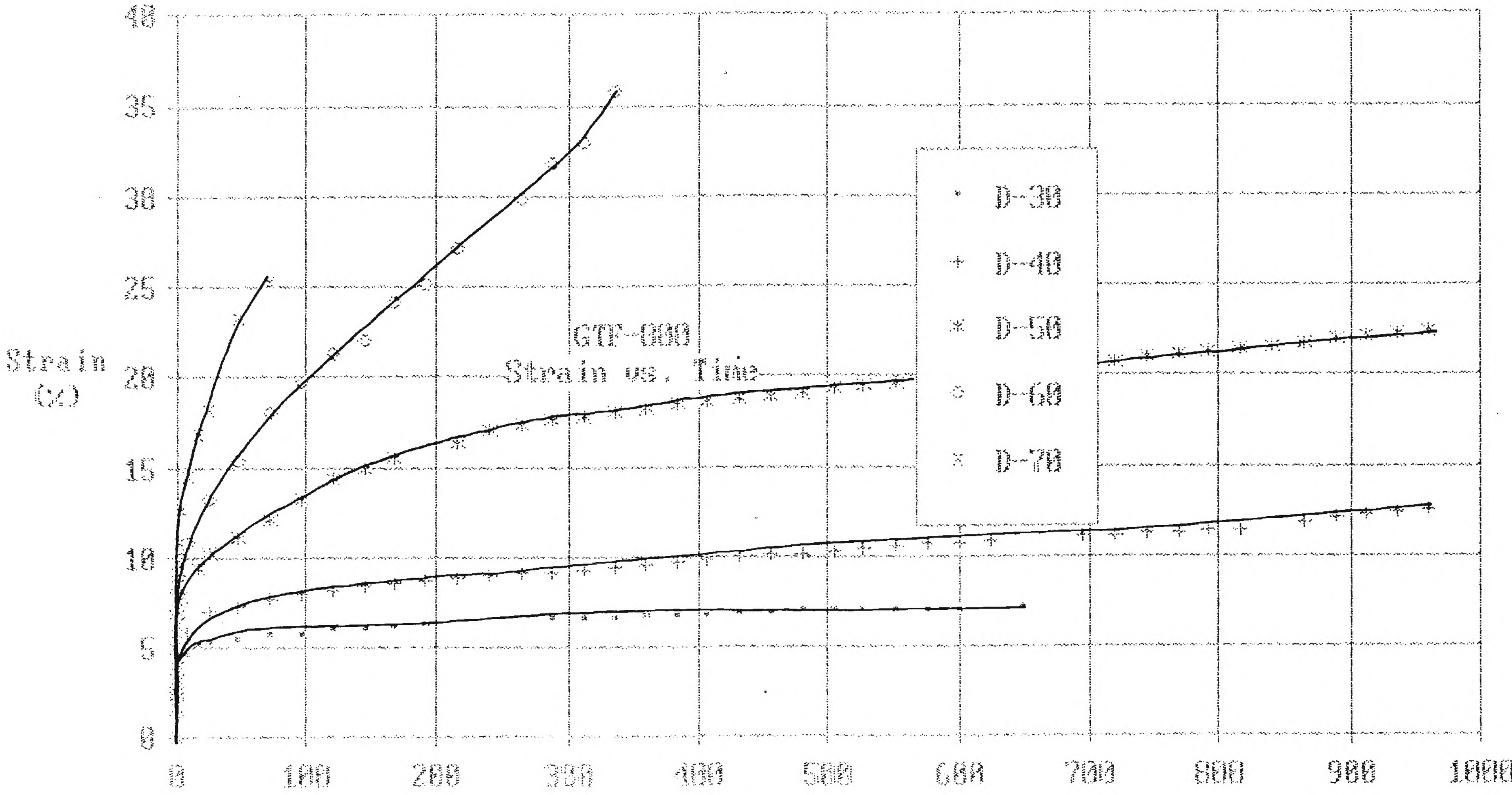
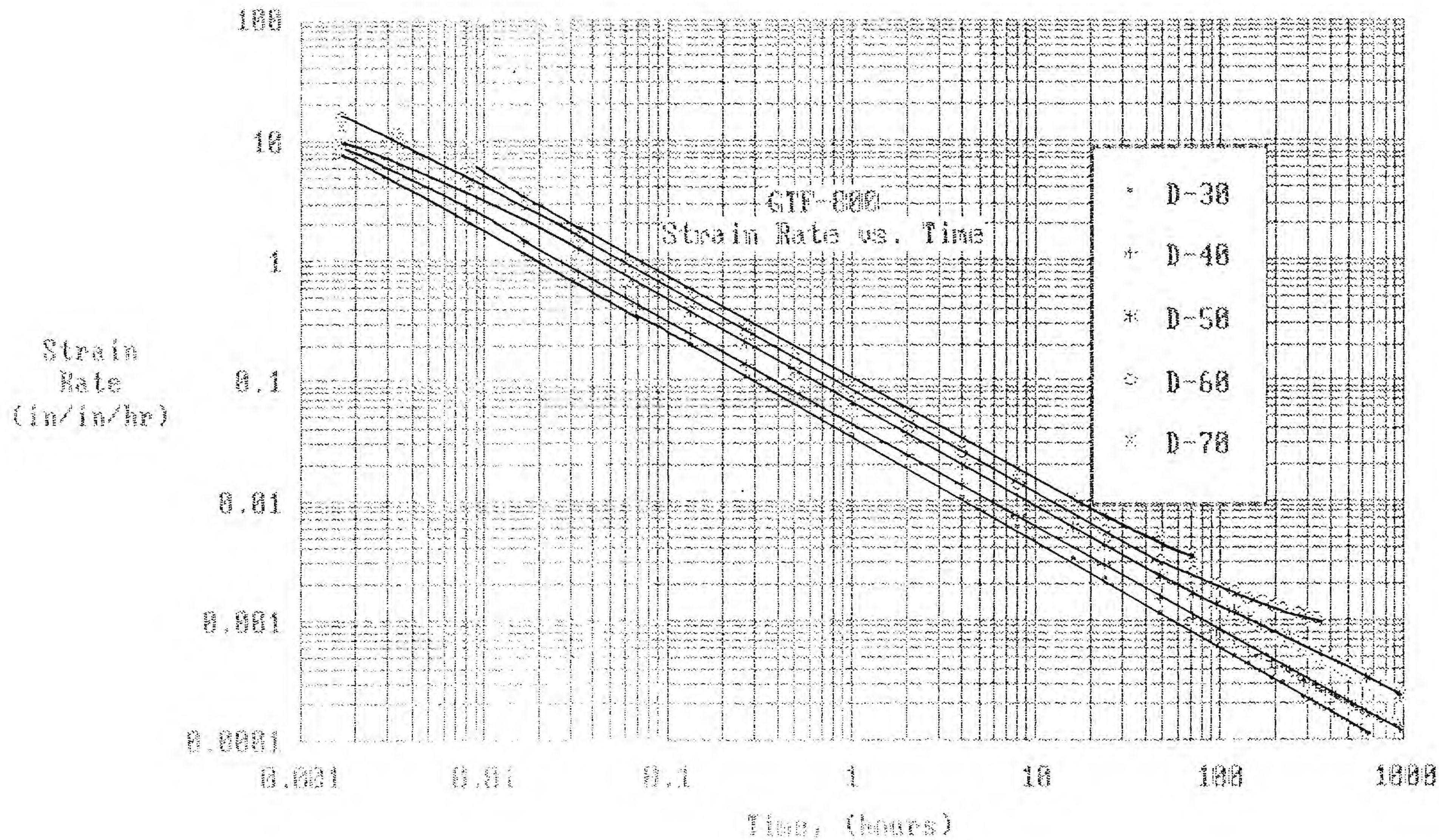
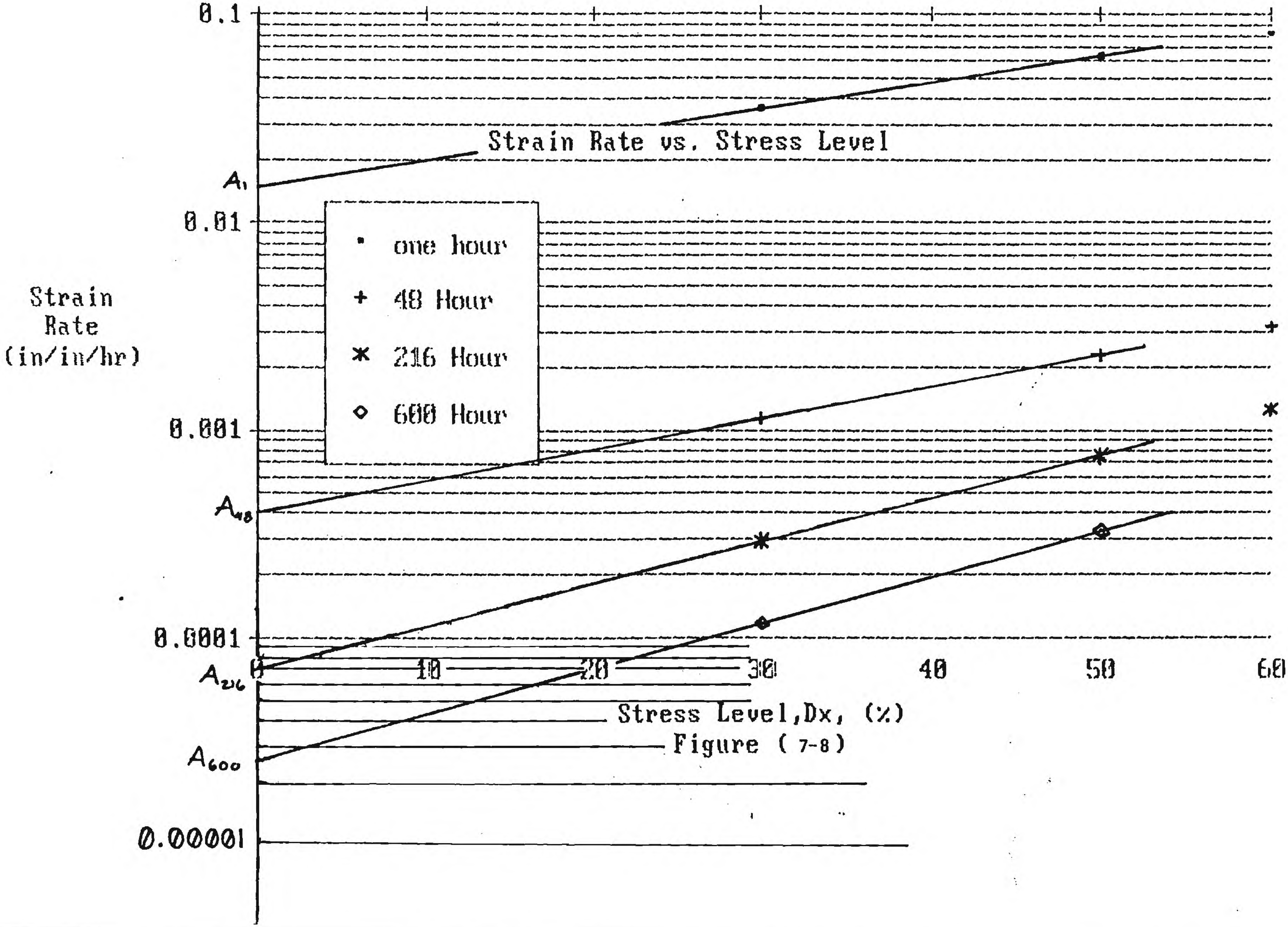

GTP-000
Strain vs. Time
Strain (%)
Time, (hours)
D-30
D-40
D-50
D-60
D-70
0
5
10
15
20
25
30
35
40
0
100
200
300
400
500
600
700
800
900
1000

Figure (7-6)


GTF-800
Strain Rate vs. Time
D-30
D-40
D-50
D-60
D-70
Strain
Rate
(in/in/hr)
100
10
1
0.1
0.01
0.001
0.0001
0.001
0.01
0.1
1
10
100
1000
Time, (hours)
Figure (7-7)



Strain Rate vs. Stress Level
0.1
0.01
0.001
0.0001
0.00001
Strain Rate (in/in/hr)
A_1
A_{48}
A_{216}
A_{600}
one hour
48 Hour
216 Hour
600 Hour
0
10
20
30
40
50
60
Stress Level,Dx, (%)
Figure (7-8)

CHAPTER 8

DESIGN PROCEDURE USING GEOTEXTILES

In the design of soil reinforced structures, the need for design methods that accurately model field conditions and long term effects is extremely important. Many design procedures are currently used which incorporate traditional soil mechanic theories together with design charts. However, most design methods do not properly account for time dependent deformations, soil/geosynthetic deformation compatibility, material degradation and construction induced damage.

A design method (4) is discussed followed by a design example using the four layer stitch-bonded composite investigated in this study. Since data available on the stitch-bonded geotextile are limited, assumptions must be made in order to present the use of this design method.

A soil reinforced structure is considered to be permenant if it built to specifications provided by the U.S. Federal Highway Administration (FHWA) for permenant highway construction. These specifications mean that a structure has a minimum design life of 75 to 100 years. A soil reinforced structure is considered critical if there is mobilized tension in the reinforcement throughout the life of the structure and if failure of the structure would cause loss of life and/or significant property damage.

Methodology

The methodology for selection of product-specific long-term reinforcement properties described herein relies on the use of constant-load creep test as described by McGown et al.(17) and on interpretation of the test results in terms of isochronous (constant-time) parameters

(1,5,17). It further requires consideration of the long-term, in-ground durability of the reinforcement as well as the effect of construction-induced material damage. The end result is a product-specific long-term reinforcement tension that can be used in limit equilibrium calculations. Both reinforcement rupture (limit state) and reinforcement load-elongation response (serviceability state) are considered. A flow chart outlining the methodology is given in Figure (8-1) (4).

Reinforcement Creep

A tension-strain curve must be established for the reinforcement material which corresponds to its behavior in a confined in-soil loading condition for the design life of the soil structure. Ideally, this curve would be determined from the results of in-soil constant-load creep tests carried out for a duration long enough to allow the extrapolation of the results to the design life of the structure. Given that polymer creep test results should not be extrapolated more than about one order of magnitude; this type of testing is not practical since the equipment is complex and a test duration of about ten years would be required. However, creep testing for any product should never be performed with a duration of less than 1,000 hours (4).

The results from creep tests can be presented in terms of isochronous (constant-time) tension-strain curves as shown in Figure (8-1). Isochronous curves can be extrapolated, either mathematically (1,17), or based on judgment, to the design life of the soil structure. Extrapolation can be carried out with a reasonable degree of confidance only if the relationship between load, strain, and log-time is linear. Above the limit of linear viscoelasticity, extrapolation is difficult (4).

Method - The tension-strain behavior for the geosynthetic should be determined using constant-load creep tests for a duration of 10,000 hours and a range of stress levels. Stress levels used are based on wide width test data as per ASTM D4595-86. Test specimens should be of sufficient size and shape as to minimize scale and shape effects.

Based on the creep test results, an extrapolated isochronous tension-strain curve should be defined corresponding to the anticipated service temperature and design life. From this isochronous curve the following two parameters can be determined:

1) The reinforcement tension at the strain corresponding to the limit state of the soil-reinforcement system. This value of tension is termed the limit state reinforcement tension and designated TL. The strain corresponding to TL should be within the range of linear viscoelastic behavior for the geosynthetic; and

2) The reinforcement tension at a strain value defined by a structural serviceabilty limit. This value of tension is designated as the serviceability limit state reinforcement tension, TS.

Based on published creep test data for different geosynthetics, TL varies from about 15% to 70% of wide width tensile strength as per ASTM D4595-86 (4)

Long-Term Material Degradation

The isochronous tension-strain relation previously mentioned does not account for material degradation. Degradation of the geosynthetic may result from internal aging of the polymer microstructure, chemical and microorganism attack, water adsorption, ultraviolet radiation, thermal oxidation, and enviromental stress cracking (4). The relative contributions of each mode of degradation depends primarily on polymer type, resin grade, and the presence of additives such as carbon black and anti-oxidants.

Method - In order to evaluate the factors affecting material degradation, product-specific studies (both literature reviews and laboratory tests) should be performed. The results from these studies should be incorporated into two durability factors of safety:

1) F.D.L equal to the durability factor of safety on the limit state reinforcement tension;

2) F.D.S equal to the durability factor of safety on the serviceability state reinforcement tension;

The above factors should be determined for the range of soil enviroments to which the reinforcement may potentially be exposed. Based on published information on different polymer types, grades and additive packages, the durability factor of safety will range from 1.0 to more than 2.0 (4).

Construction Induced Damage

In the design of any soil-reinforced structure, the effects of construction operations must be considered. Performing laboratory in-soil creep tests subjected to project-specific fill placement and compaction procedures is not practical. Instead, test data from full scale construction damage tests should be performed. Such tests should use large size specimens with fill placement and compaction performed with standard construction equipment and fill lift thicknesses. Comparison of wide width data as per ASTM D4595-86 before and after placement of the specimens is used to evaluate the effects of construction-induced damage.

Method - The effects of construction-induced damage is determined from full scale tests using fill materials, placement and compaction procedures, and equipment representative of the anticipated site conditions. The results from such testing are then converted into construction damage factors of safety:

1) F.C.L equal to the factor of safety on the limit state reinforcement tension; and,

2) F.C.S equal to the factor of safety on the serviceability state reinforcement tension.

Based on published information for different geosynthetics, the construction damage factor of safety will range from 1.0 to 1.7 (4).

Long-Term Reinforcement Tension

The long-term allowable reinforcement tension, accounting for creep, time dependent degradation and construction-induced damage, is taken as the lesser of:

1) Limit state determination.

The long term reinforcement tension based on the load limited criterion, TAL, is given by

$$TAL = \frac{TL}{F.D.L \times F.C.L \times F.S.} \qquad (8\text{-}1)$$

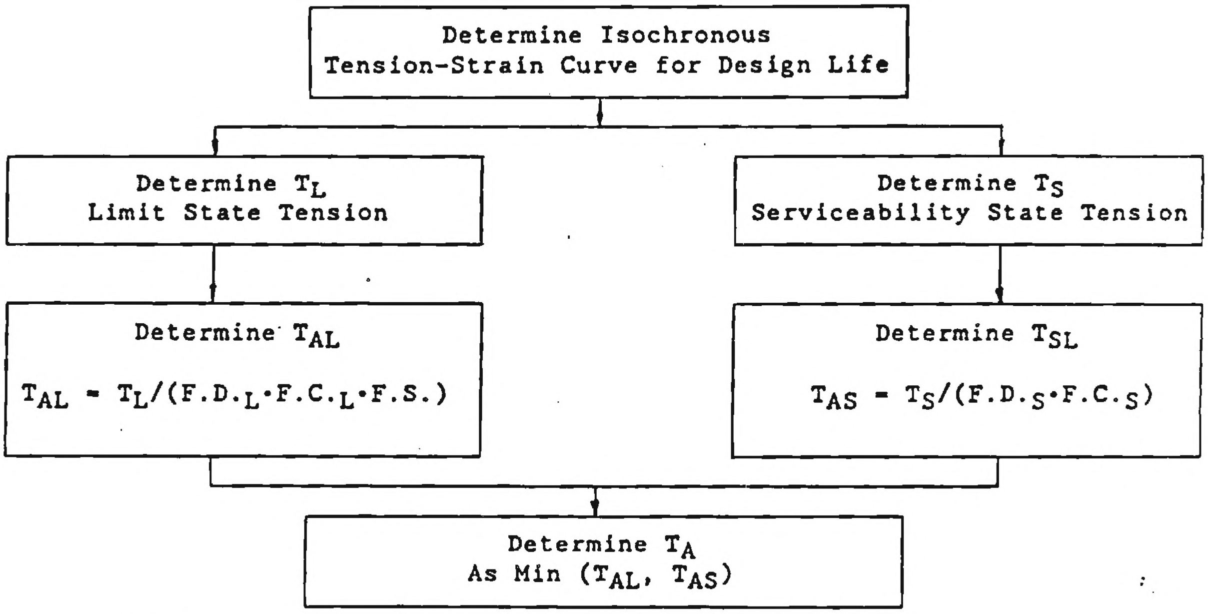
where F.S. is an overall factor of safety against internal failure of the soil structure. Typical values for F.S. are 1.3 to 1.5 for reinforced slopes and 1.5 for retaining walls.

2) Serviceability state determination.

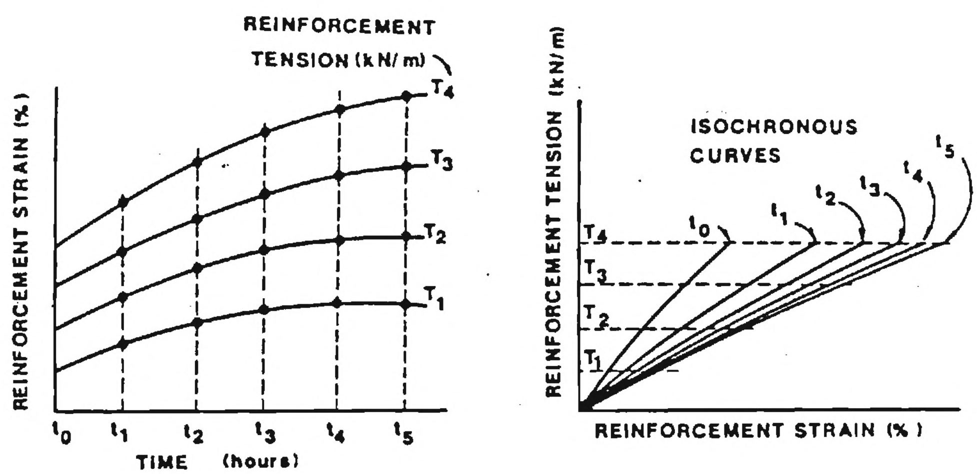
The long-term reinforcement tension based on the serviceability criterion, TAS, is given by

$$TAS = \frac{TS}{F.D.S \times F.C.S} \qquad (8\text{-}2)$$

Since this is a working stress calculation, no overall factor of safety is required. F.D.S and F.C.S may be smaller than F.D.L and F.C.L since limited data (4) suggests that long-term degradation and construction damage have a larger effect on peak strength than on initial modulus (4).



8-1a Flow chart



8-1b Derivation of isochronous tension-strain curves from constant-load creep test results (after McGown et al.)

Figure 8-1

CHAPTER 9

APPLICATION OF METHODOLOGY

The following example is given to present the methodology described above. Data from the tension creep tests performed for this study are used in the design example. However, assumptions are made with regard to degradation and construction damage due to the lack of field data available for the GTF-800 product.

Reinforcement Creep

From Figure (8-2), it is seen that for values of stress level less than about 35%, the Strain rate versus LogTime response of the GTF-800 is approximately linear. At the 50% stress level, the response is nonlinear. Based on these results, extrapolation of the isochronous curves to 10,000 hours should be limited to stress levels less than about 35%. The isochronous load-strain curve for 10,000 hours obtained by linear extrapolation of the creep test results is shown in Figure (8-3).

McGown et al. suggests a "performance limit strain" of 10% to be used for soil reinforcement systems. Bonaparte et al. suggest a "serviceability state strain" of 5% based on experience and judgement.

For limit state and serviceability state strains of 10% and 5%, respectively, TL and TS are found from Figure (8-3). From Figure (8-3), TS and TL are found to be 1035 lbs/ft and 2100 lbs/ft, respectively.

Long-Term Material Degradation

The factors influencing in-ground material degradation of geosynthetics are generally site specific. Therefore, reproducing field conditions in the laboratory is extremely important.

Based on information provided by Bonaparte et al. and in order to continue the example, the factors of safety with respect to durability, F.D.L and F.D.S are both taken to be 1.0.

Construction-Induced Damage

Full scale construction damage tests are necessary to properly evaluate the safety factors with respect to construction damage. Bonaparte et al. suggest a value of 1.15 for F.C.L. and 1.0 for F.C.S..

Long-Term Reinforcement Tension

The allowable long-term reinforcement tension for the GTF-800 is evaluated using the methodology previously described.

Assuming the soil-structure is to be a concrete-panel-faced retaining wall, for example, and using crushed stone as back fill with a maximum particle size of 2.5 inches, the following calculations are carried out:

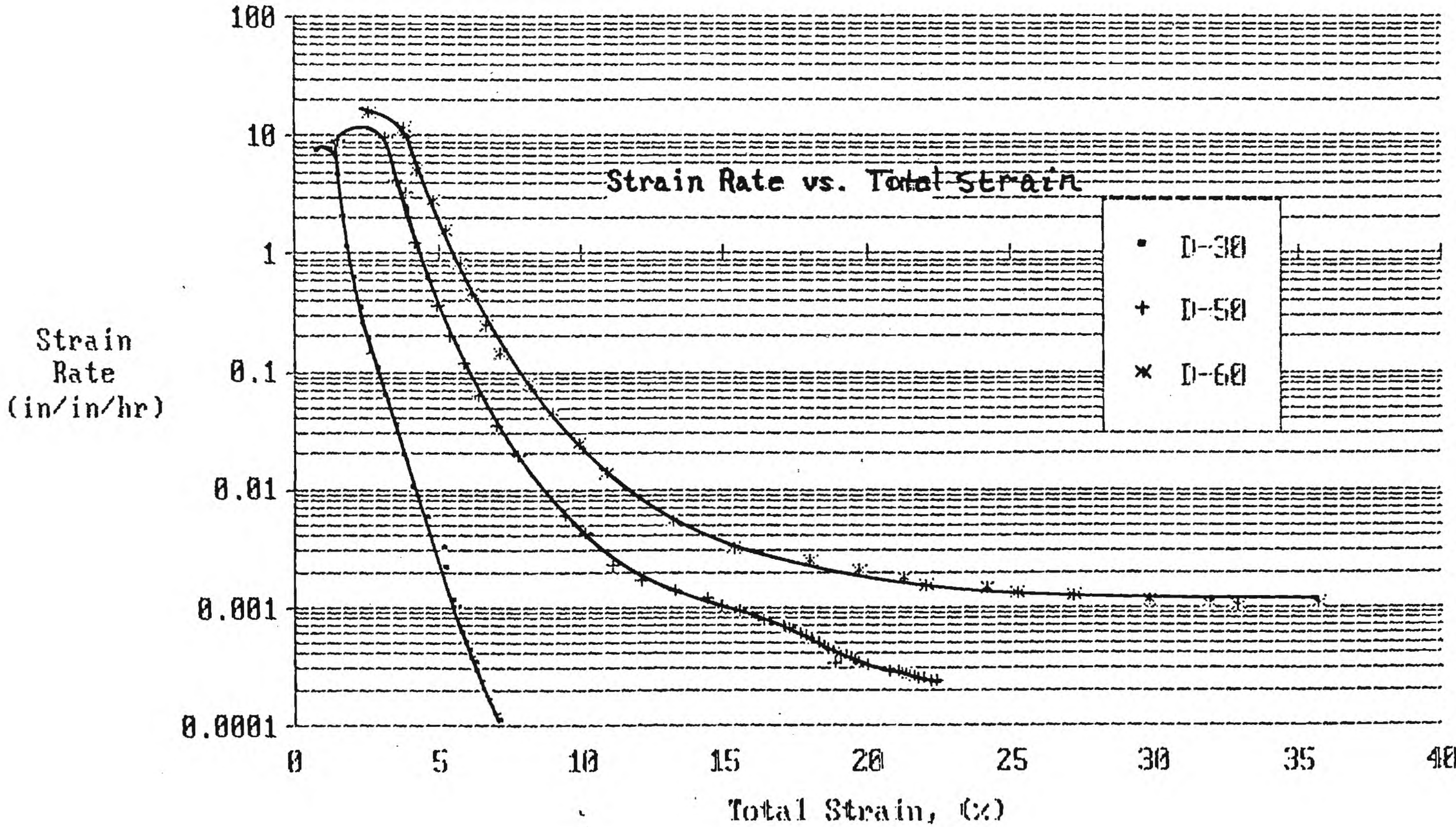
$$TAL = \frac{TL}{F.D.L. \times F.C.L. \times F.S.} = \frac{2100 \text{ lbs/ft}}{1.0 \times 1.15 \times 1.5} = 1217 \text{ lbs/ft}$$

$$TAS = \frac{TS}{F.D.S. \times F.C.S.} = \frac{1035 \text{ lbs/ft}}{1.0 \times 1.0} = 1035 \text{ lbs/ft}$$

Taking the minimum of TAL and TAS:

$$TA = TAS = 1035 \text{ lbs/ft}$$

Therefore, the value of allowable long-term reinforcement tension for use in equilibrium equations is approximately 16 % of the GTF-800 products' wide width strength of 6563 lbs/ft. From these findings, the issue of safe design using index test results is plain.



Strain Rate vs. Total Strain
D-30
D-50
D-60
Strain Rate (in/in/hr)
100
10
1
0.1
0.01
0.001
0.0001
0
5
10
15
20
25
30
35
40
Total Strain, (%)

Figure (8-2)

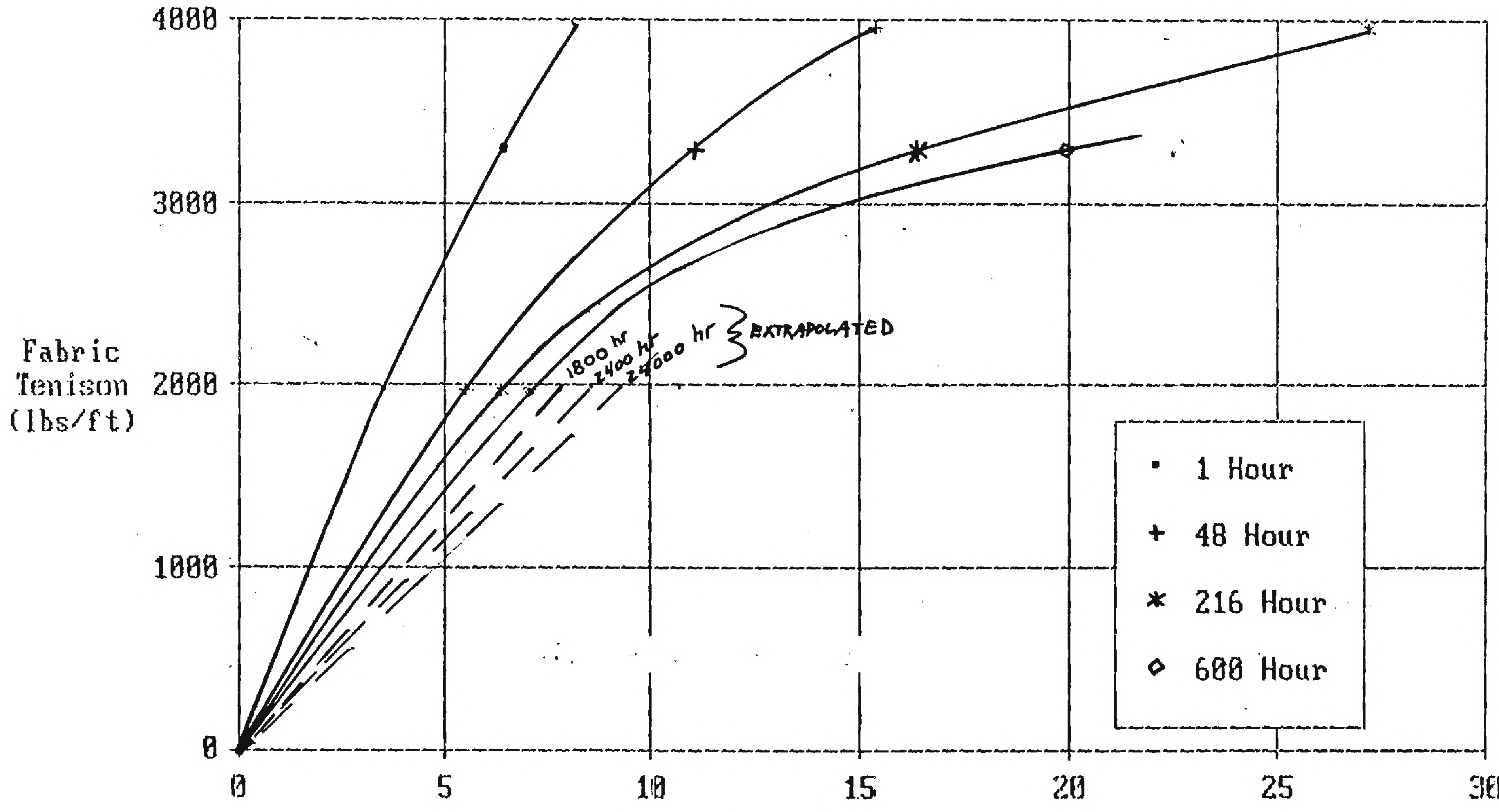

Fabric
Tenison
(lbs/ft)
4000
3000
2000
1000
0
0
5
10
15
20
25
30
1800 hr
2400 hr
24000 hr
EXTRAPOLATED
1 Hour
48 Hour
216 Hour
600 Hour
Total Strain, (%)

Figure (8-3)

CHAPTER 10

CONCLUSIONS

The purpose of this study has been to investigate the material properties of stitch-bonded geotextiles with respect to paramters needed for soil-reinforcement design. Interface friction parameters, confined and unconfined tensile strength, and unconfined tension creep relations have been presented. In addition, a methodology for estimating the allowable long-term reinforcement tension to be used in design has been given along with a limited example using the GTF-800 product.

The results from the direct shear program suggest that the stitch-bonded composites are slightly sensitive to the number of layers used with respect to the coefficient of friction, interface friction angle and adhesion. This is true for both the sand and sand/clay mixture with the sand/clay mixture yielding slightly lower values except for the woven/nonwoven composite.

The results of the wide width program show the GTF-800 product to have an unconfined tensile strength of approximately 6563 pound per foot of width. The results of the confined tensile strength program show the GTF-800 product to have a confined tensile strength of approximately 7000 pounds per foot of width. When comparing the results of these two testing programs, the effects of confinement on the GTF-800 product can be stated as being minimal. However, a slight increase was indeed observed. This increase is important in that some geosynthetic products undergo a significant decrease in strength when confined, especially in the case where the confining soil medium is of a large size and angularity. Further research is strongly incouraged in the area of the effects of confining medium on tensile strength.

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APPENDIX A.

LSPCD USERS MANUAL

APPENDIX A: USER'S MANUAL FOR DATA COLLECTION PROGRAMS

Two computer programs were used to collect the data generated in operating the LSPCD; PULLGRID.BAS and PULLTEXT.BAS.

PULLGRID is a basic program used to collect force and displacement data when operating the LSPCD on a geogrid sample. This program determines the force component, Alpha, as:

$$\text{Alpha} = \frac{\text{LBS} * \text{SF}}{\text{SS}}$$

where: Alpha = horizontal force on the geogrid per foot of width;

LBS = horizontal load in pounds;

SF = number of longitudinal strands per foot; and,

SS = number of longitudinal strands per sample.

Through this equation, Alpha is normalized based on the distribution of longitudinal strands in a sample.

A separate program was written to evaluate geotextiles in the LSPCD. This program is designated as PULLTEXT.BAS, where Alpha is calculated as:

$$\text{Alpha} = \frac{\text{LBS} * 12}{\text{W}}$$

where: Alpha = horizontal force on the geotextile per foot;

LBS = horizontal load in pounds; and,

W = width of geotextile in inches.

Program Operation

The operational proceedure for using the two data collection programs is as follows:

1. Boot the computer with the specialized MS-DOS disk capable of operating on Drive C: with 520k memory (Pullout Disk # 1, LSPCD Start-up DOS).

2. Open Drive C by typing: A > C:

3. Copy BASICA.COM onto Drive C: C > COPY A: BASICA.COM

4. Replace the DOS disk with the data collection program disk (PULLOUT DISK # 2, LSPCD Data Collection Programs).

5. Copy one of the data collection programs onto Drive C: Example: if running a geogrid test type:

C > COPY A:PULLGRID.BAS

6. Activate BASICA by typing:

C > BASICA

7. Load the PULLGRID program by depressing the F3 function key and typing:

load "PULLGRID"

8. Execute the program by depressing the F2 function key.

9. The program will request the following parameters depending on the sample tested:

Geogrid Test	Geotextile Test
Output Filename	Output Filename
Number of Strands per Sample	Specimen Width (in.)
Number of Strands per Foot	Delay in Seconds Between
Delay in Seconds Between	Sampling Time(HH:MM:SS:)
Sampling Time:(HH:MM:SS:)	

The output filename is given as PULLN.DAT where N refers to the test number.

10. Values displayed on the screen are in the following order:

Horizontal Displacement Rate, Horizontal Displacement, Load, Alpha, Time

11. The LVDT and the Load Cell are adjusted to approximately read zero on the screen.

12. Prior to the start of a test, the F4 function key is depressed to save test data. Important: On screen data will not be saved unless the F4 key is punched.

13. At the conclusion of a test, the data collection is siezed by holding down the CTRL and BREAK keys simultaneously. To exit BASICA type: SYSTEM

14. The data output file is stored on a floppy disk inserted in drive A by typing:

```
C> COPY PULLN.DAT A:
```

15. A data reduction procedure is then conducted in order to evaluate the data output. This proceedure is explained in Appendix B.

A listing of the two data collection programs is given on the following pages.

```
10 REM
20 P=0:Q=0
30 CLS
40       INPUT "OUTPUTFILE";FIL$
50       OPEN FIL$ FOR OUTPUT AS #1
60       INPUT "SPECIMEN WIDTH (IN.)";W
70       INPUT "DELAY IN SECONDS BETWEEN SAMPLING";S
80       ADDRESS = 784
90       S=S*5/6
100       A=0
110       INPUT "TIME(HH:MM:SS) =";T$
120       TIME$=T$
130 FOR I=0 TO 1 STEP 1
140       IF A=0  THEN GOSUB 490
150       A=A+1
160 REM disable auto-incrementing, external start conversions, and all
170 REM interupts.  gain=1.
180 OUT ADDRESS+4,128
190 REM output channel number
200 OUT ADDRESS+5,I
210 REM start a conversion.
220 OUT ADDRESS+6,0
230 REM wait until bit 7 of status b te is a 1 signaling done converting
240 REM read in the data .
250 LOW = INP( ADDRESS + 5 )
260 HIGH = INP( ADDRESS + 6 )
270 REM convert from twos complement to a number between -10 and 10
280 X = 256*HIGH + LOW
290 IF X > 32767 THEN X = X-65536!
300 VOLTAGE = X/204.8
310      IF I = 1 GOTO 330
320      DEFL = (VOLTAGE * - .10169)+1
330      LBS = (VOLTAGE -.024)*1148.02
340      NEXT I
350      A$=INKEY$
360      IF A$="W" THEN Q=1:GOSUB 540:TIME$="00:00:0"
370      IF A$="S" THEN P=1
380     IF A$="F" THEN 570
390      ALPHA=LBS*12/W
400 STRRAT=((DEFL-PDEFL)/(PTIME/TIMER))*60
410      P$="          #####.####    #####.####       #####.####      #####.###
411      P1$="#.#######   #####.####    #####.####       #####.####      #####.##

420      PRINT USING P1$;STRRAT,DEFL,LBS,ALPHA,TIMER
430 PDEFL=DEFL:PTIME=TIMER
440      IF Q=1 THEN LPRINT USING P$;DEFL,LBS,ALPHA,TIMER
450     IF P=1 THEN WRITE#1,DEFL,LBS,ALPHA,TIMER
460      FOR I = 1 TO S*1000:NEXT I
470      IF A=104 THEN A=0
480      GOTO 130
490     REM SUBROUTINE SPACE
500     IF Q<>1 THEN GOTO 560
510     LPRINT CHR$(15)
520     LPRINT
530     LPRINT
540     LPRINT "          LVDT (IN)      LOAD (LBS)     ALPHA (#/FT)        TIME"
550     LPRINT
560     RETURN
570   END
```

APPENDIX B: USER'S MANUAL FOR DATA REDUCTION PROCEDURE

Three computer programs are used to reduce the data output file; CLAMPG.FOR, CLAMPT.FOR AND ZEROOUT.FOR. CLAMPG.FOR is a clamp calibration program set up for geogrids. This program calculates Alpha in pounds per foot of width. CLAMPT.FOR is to be used for calibration when testing a geotextile. This program calculates Alpha in pounds per foot. CLAMPT.FOR is run in the same manner as CLAMPG.FOR and therefore is not referred to in the Data Reduction Procedure. ZEROOUT.FOR is independent of Alpha.

Data Reduction Procedure

1. Reduce PULLN.DAT to < 130 data points in order to load the datafile onto Microsoft Chart. The deletion of data can be done using a word processing/editor program or EDLIN. This new file is called PULLN.RED and saved on the datafile disk.

2. Create blank output files to be used later in the data reduction process. These files are called PULLN.CAL and PULLN.ZER and stored on the datafile disk.

3. Calibrate the data for the clamp effect using the program CLAMPG.FOR (use CLAMPT.FOR if testing a geotextile). Insert the data reduction program disk (PULLOUT DISK # 3) into Drive A and the datafile disk into Drive B and type

```
A> CLAMPG
```

The program will request the following input parameters:

ENTER DATA FILENAME ==> B:PULLN.RED

ENTER OUTPUT FILENAME ==> B:PULLN.CAL

ENTER NORMAL PRESSURE AS 0,1,3,5 or 10 PSI ==>

ENTER TOTAL NUMBER OF DATA POINTS (N<130) ==>

After running the calibration program, examine the datafile PULLN.CAL for initial negative Alpha values due to the clamp effect. Remove these datapoints and record the new total number of datapoints.

4. Zero the calibrated datafile for initial dïsplacement and alpha values using the program ZEROOUT.FOR and run the program by typing:

A > ZEROOUT

The program will request the following input parameters:

ENTER DATA FILENAME ==> B: PULLN.CAL

ENTER OUTPUT FILENAME ==> B: PULLN.ZER

ENTER TOTAL NUMBER OF DATAPOINTS (N<130) ==>

5. Load PULLN.ZER onto Microsoft Chart software to view data graphically.

The three data reduction programs are listed on the following pages.

```
      PROGRAM CLAMPG
      DIMENSION A(130),B(130),C(130),D(130),E(130)
      CHARACTER FIN*15,FOUT*15
      WRITE(*,10)
 10   FORMAT('       ENTER DATA FILENAME==> ':)
      READ(*,30) FIN
      WRITE(*,20)
 20   FORMAT('     ENTER OUTPUT FILENAME==> ':)
      READ(*,30) FOUT
 30   FORMAT(A)
      OPEN(UNIT=5,FILE=FIN)
      OPEN(UNIT=6,FILE=FOUT)
      WRITE(*,40)
 40   FORMAT('       ENTER NORMAL PRESSURE AS 0, 1, 3, 5, OR 10 ==> ':)
      READ(*,*) NP
      WRITE(*,50)
 50   FORMAT('  ENTER TOTAL NUMBER OF DATA POINTS (N<130) ==> ':)
      READ(*,*) N
      DO 15 I=1,N
      READ(5,*) A(I),B(I),C(I),D(I)
 15   CONTINUE
      IF (NP.EQ.5) GOTO 100
      IF (NP.EQ.0) GOTO 200
      IF (NP.EQ.1) GOTO 300
      IF (NP.EQ.3) GOTO 400
      DO 25 I=1,N
      IF (A(I).LT.0.063) THEN
       E(I)=C(I)-A(I)*11755
      ELSE IF (A(I).GT.1.2) THEN
       E(I)=C(I)-482
      ELSE
       E(I)=C(I)-(755-A(I)*227)
      END IF
 25   CONTINUE
      GOTO 900
100   DO 35 I=1,N
      IF (A(I).LT.0.033) THEN
       E(I)=C(I)-A(I)*9394
      ELSE IF (A(I).LT.0.10.AND.A(I).GT.0.033) THEN
       E(I)=C(I)-(288+A(I)*681)
      ELSE IF (A(I).GT.1.0) THEN
       E(I)=C(I)-(322-A(I)*69)
      ELSE
       E(I)=C(I)-(369-A(I)*140)
      END IF
 35   CONTINUE
      GOTO 900
200   DO 45 I=1,N
      E(I)=C(I)-86
 45   CONTINUE
      GOTO 900
300   DO 55 I=1,N
      IF (A(I).LT.0.081) THEN
       E(I)=C(I)-A(I)*1383
      ELSE IF (A(I).GT.0.543) THEN
       E(I)=C(I)-(99-A(I)*20)
      ELSE
       E(I)=C(I)-(116-A(I)*52)
      END IF
 55   CONTINUE
      GOTO 900
400   DO 65 I=1,N
      IF (A(I).LT.0.080) THEN
       E(I)=C(I)-A(I)*3363
```

```
      E(I)=C(I)-(189-A(I)*49)
      ELSE
       E(I)=C(I)-(292-A(I)*288)
      END IF
 65   CONTINUE
900   DO 95 I=1,N
      WRITE(6,60) A(I),C(I),E(I)
 95   CONTINUE
 60   FORMAT(1X,F11.8,',',F10.4,',',F10.4)
      STOP
      END
```

```
      PROGRAM CLAMPT
      DIMENSION A(130),B(130),C(130),D(130),E(130)
      CHARACTER FIN*15,FOUT*15
      WRITE(*,10)
 10   FORMAT('        ENTER DATA FILENAME==> ':)
      READ(*,30) FIN
      WRITE(*,20)
 20   FORMAT('      ENTER OUTPUT FILENAME==> ':)
      READ(*,30) FOUT
 30   FORMAT(A)
      OPEN(UNIT=5,FILE=FIN)
      OPEN(UNIT=6,FILE=FOUT)
      WRITE(*,40)
 40   FORMAT('        ENTER NORMAL PRESSURE AS 0, 1, 3, 5, OR 10 ==> ':)
      READ(*,*) NP
      WRITE(*,50)
 50   FORMAT('  ENTER TOTAL NUMBER OF DATA POINTS (N<130) ==> ':)
      READ(*,*) N
      DO 15 I=1,N
      READ(5,*) A(I),B(I),C(I),D(I)
 15   CONTINUE
      IF (NP.EQ.5) GOTO 100
      IF (NP.EQ.0) GOTO 200
      IF (NP.EQ.1) GOTO 300
      IF (NP.EQ.3) GOTO 400
      DO 25 I=1,N
      IF (A(I).LT.0.063) THEN
       E(I)=C(I)-A(I)*13651
      ELSE IF (A(I).GT.1.2) THEN
       E(I)=C(I)-560
      ELSE
       E(I)=C(I)-(877-A(I)*264)
      END IF
 25   CONTINUE
      GOTO 900
100   DO 35 I=1,N
      IF (A(I).LT.0.033) THEN
       E(I)=C(I)-A(I)*10909
      ELSE IF (A(I).LT.0.10.AND.A(I).GT.0.033) THEN
       E(I)=C(I)-(334+A(I)*791)
      ELSE IF (A(I).GT.1.0) THEN
       E(I)=C(I)-(347-A(I)*80)
      ELSE
       E(I)=C(I)-(429-A(I)*162)
      END IF
 35   CONTINUE
      GOTO 900
200   DO 45 I=1,N
```

```
300     DO 55 I=1,N
        IF (A(I).LT.0.081) THEN
         E(I)=C(I)-A(I)*1606
        ELSE IF (A(I).GT.0.543) THEN
         E(I)=C(I)-(115-A(I)*23)
        ELSE
         E(I)=C(I)-(135-A(I)*60)
        END IF
 55     CONTINUE
        GOTO 900
400     DO 65 I=1,N
        IF (A(I).LT.0.080) THEN
         E(I)=C(I)-A(I)*3905
        ELSE IF (A(I).GT.0.431) THEN
         E(I)=C(I)-(219-A(I)*57)
        ELSE
         E(I)=C(I)-(339-A(I)*334)
        END IF
 65     CONTINUE
900     DO 95 I=1,N
        WRITE(6,60) A(I),C(I),E(I)
 95     CONTINUE
 60     FORMAT(1X,F11.8,',',F10.4,',',F10.4)
        STOP
        END
```

```
        PROGRAM ZEROOUT
        DIMENSION A(130),B(130),C(130),D(130),E(130),F(130)
        CHARACTER FIN*15,FOUT*15
        WRITE(*,10)
 10     FORMAT('       ENTER DATA FILENAME==> ':)
        READ(*,30) FIN
        WRITE(*,20)
 20     FORMAT('     ENTER OUTPUT FILENAME==> ':)
        READ(*,30) FOUT
 30     FORMAT(A)
        OPEN(UNIT=5,FILE=FIN)
        OPEN(UNIT=6,FILE=FOUT)
        WRITE(*,50)
 50     FORMAT('  ENTER TOTAL NUMBER OF DATA POINTS (N<130) ==> ':)
        READ(*,*) N
        DO 15 I=1,N
        READ(5,*) A(I),B(I),C(I),
 15     CONTINUE
        DO 25 I=1,N
        D(I)=A(I)-A(1)
        E(I)=B(I)-B(1)
        F(I)=C(I)-C(1)
 25     CONTINUE
        DO 35 I=1,N
        WRITE(6,60) D(I),E(I),F(I)
 35     CONTINUE
 60     FORMAT(1X,F11.8,',',F10.4,',',F10.4)
        STOP
        END
```

APPENDIX B.

DIRECT SHEAR RESULTS

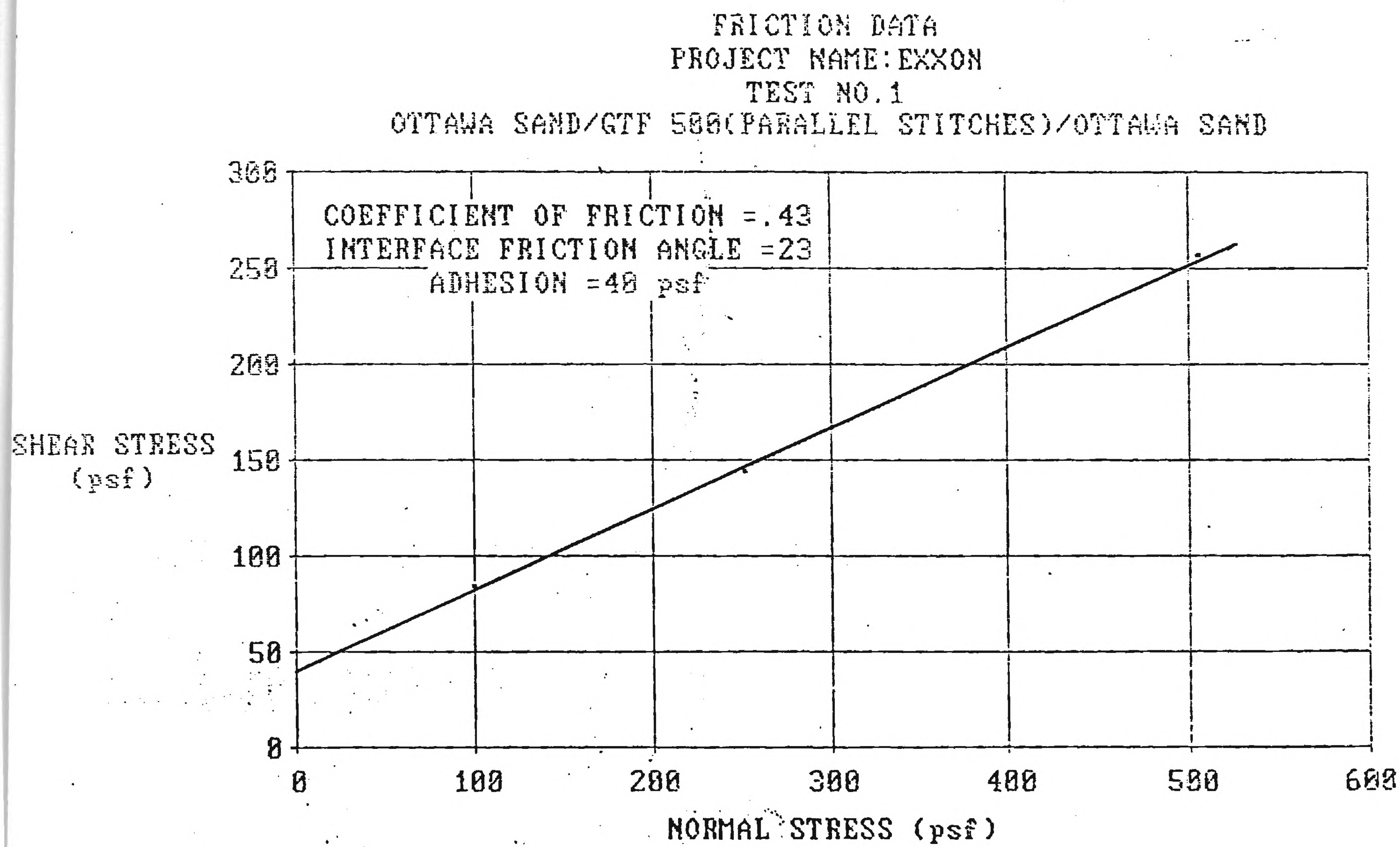

FRICTION DATA
PROJECT NAME:EXXON
TEST NO.1
OTTAWA SAND/GTF 500(PARALLEL STITCHES)/OTTAWA SAND
COEFFICIENT OF FRICTION =.43
INTERFACE FRICTION ANGLE =23
ADHESION =40 psf
SHEAR STRESS (psf)
300
250
200
150
100
50
0
0
100
200
300
400
500
600
NORMAL STRESS (psf)

Figure 4-2

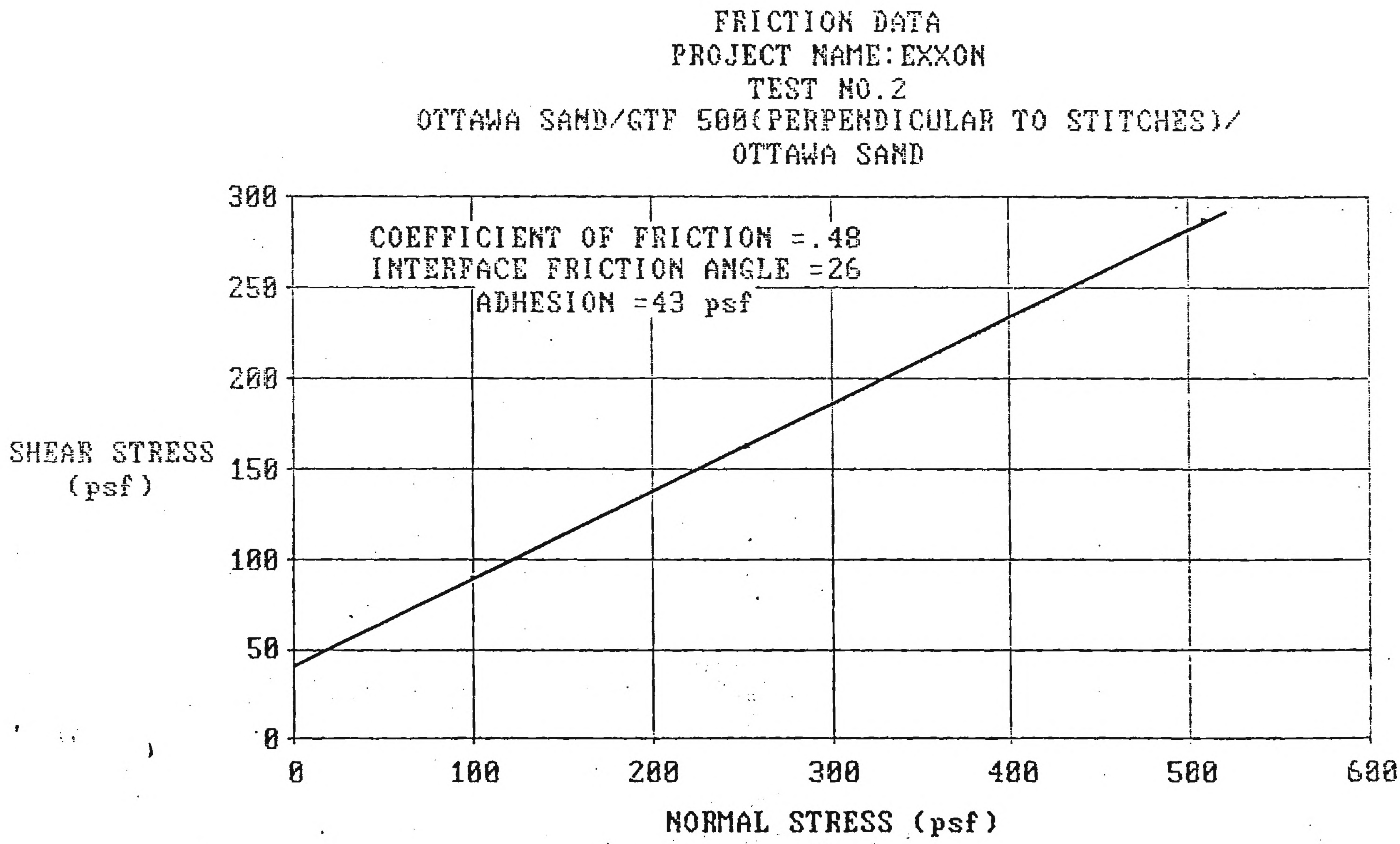
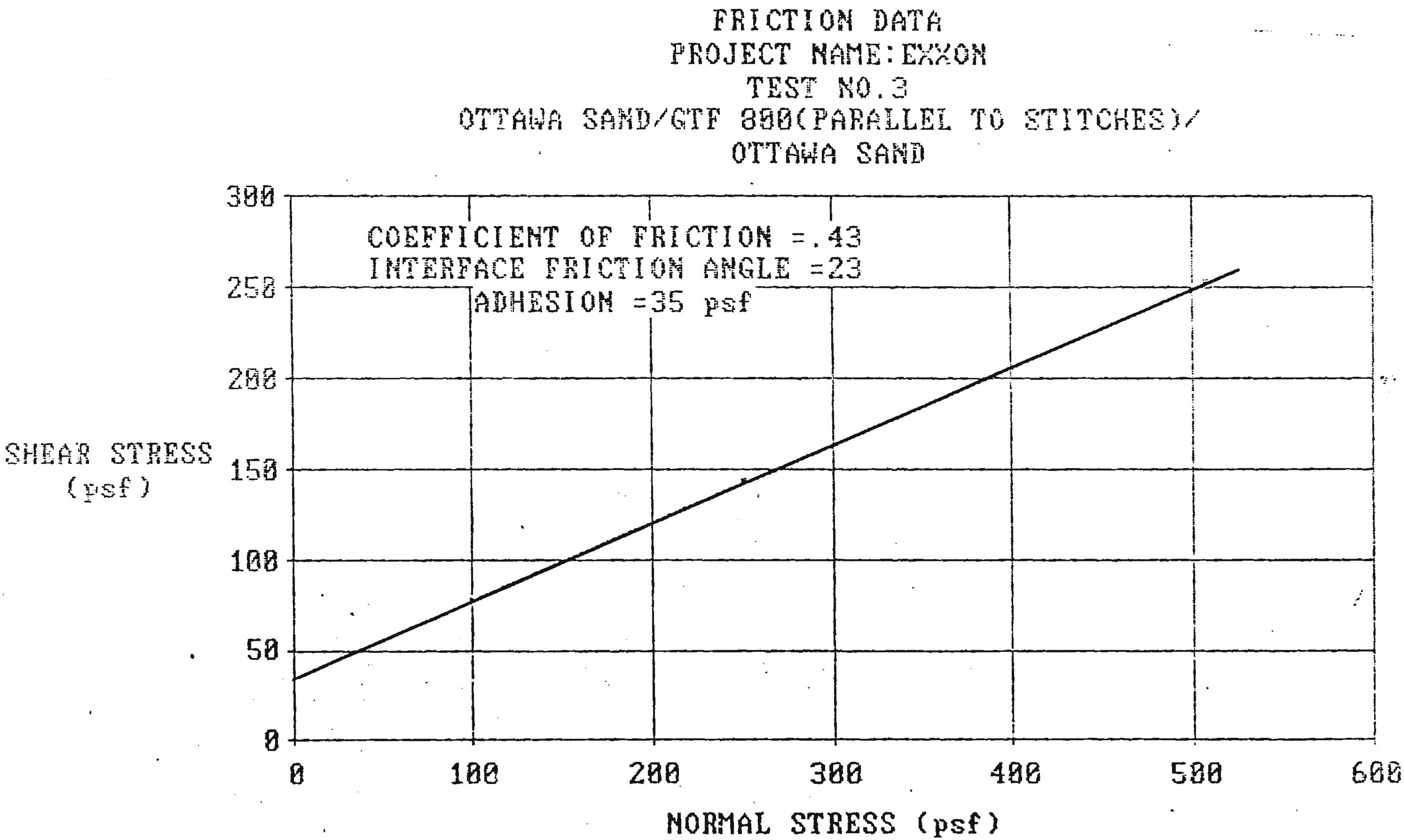

FRICTION DATA
PROJECT NAME:EXXON
TEST NO.2
OTTAWA SAND/GTF 500(PERPENDICULAR TO STITCHES)/
OTTAWA SAND
COEFFICIENT OF FRICTION =.48
INTERFACE FRICTION ANGLE =26
ADHESION =43 psf
SHEAR STRESS
(psf)
300
250
200
150
100
50
0
0
100
200
300
400
500
600
NORMAL STRESS (psf)

Figure 4-3



FRICTION DATA
PROJECT NAME:EXXON
TEST NO.3
OTTAWA SAND/GTF 800(PARALLEL TO STITCHES)/
OTTAWA SAND
COEFFICIENT OF FRICTION =.43
INTERFACE FRICTION ANGLE =23
ADHESION =35 psf
SHEAR STRESS
(psf)
300
250
200
150
100
50
0
0
100
200
300
400
500
600
NORMAL STRESS (psf)

Figure 4-4

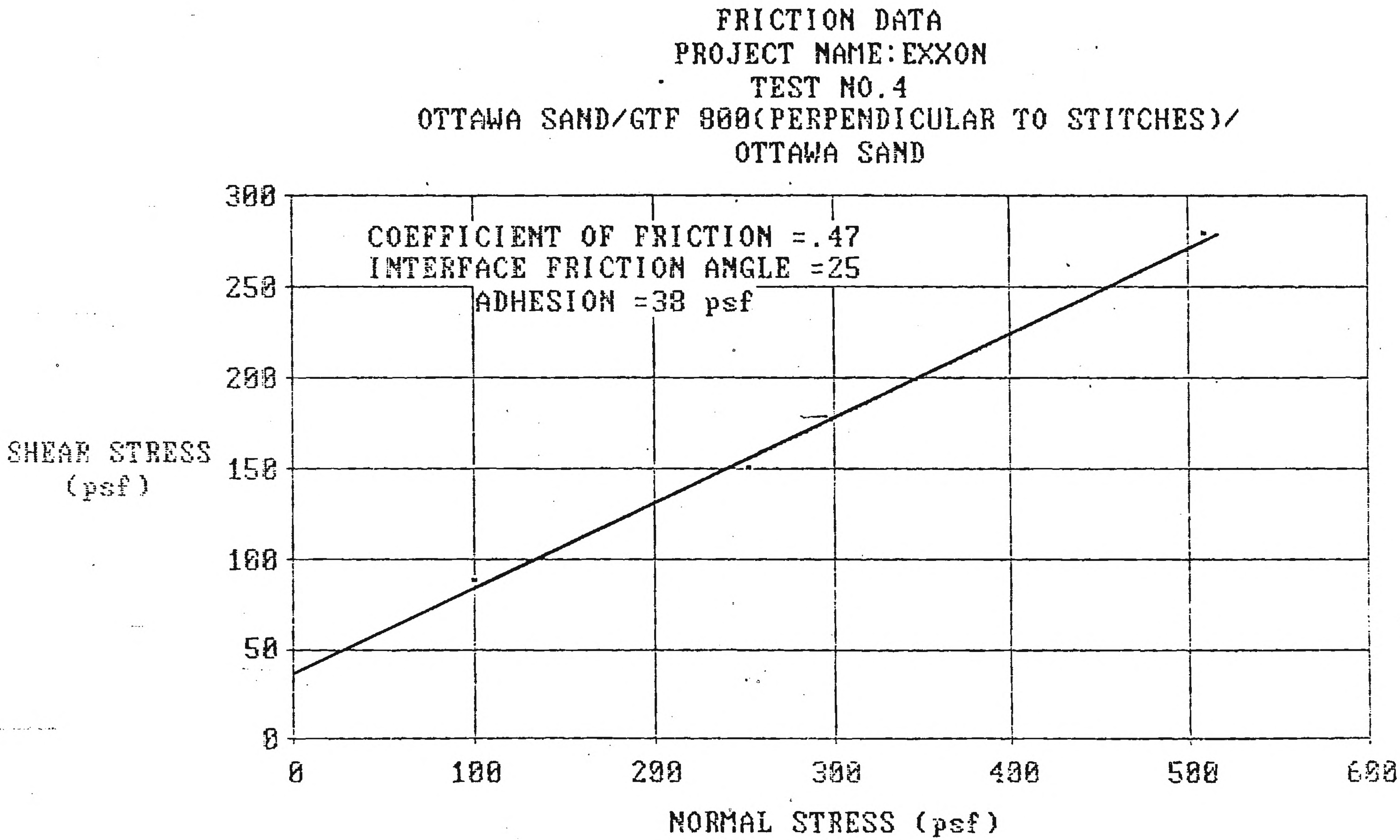

FRICTION DATA
PROJECT NAME:EXXON
TEST NO.4
OTTAWA SAND/GTF 800(PERPENDICULAR TO STITCHES)/
OTTAWA SAND
COEFFICIENT OF FRICTION =.47
INTERFACE FRICTION ANGLE =25
ADHESION =38 psf
SHEAR STRESS
(psf)
300
250
200
150
100
50
0
0
100
200
300
400
500
600
NORMAL STRESS (psf)

Figure 4-5

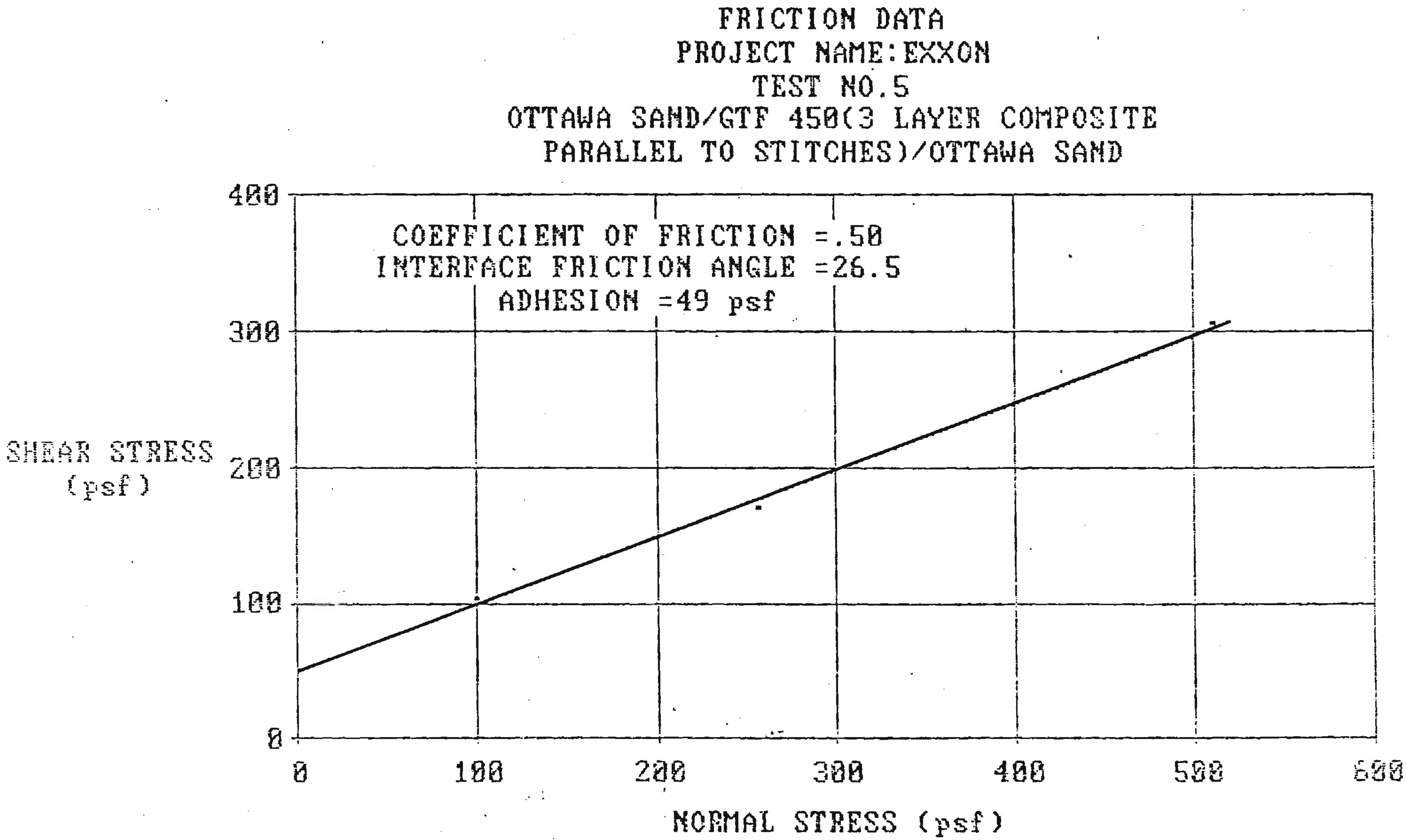
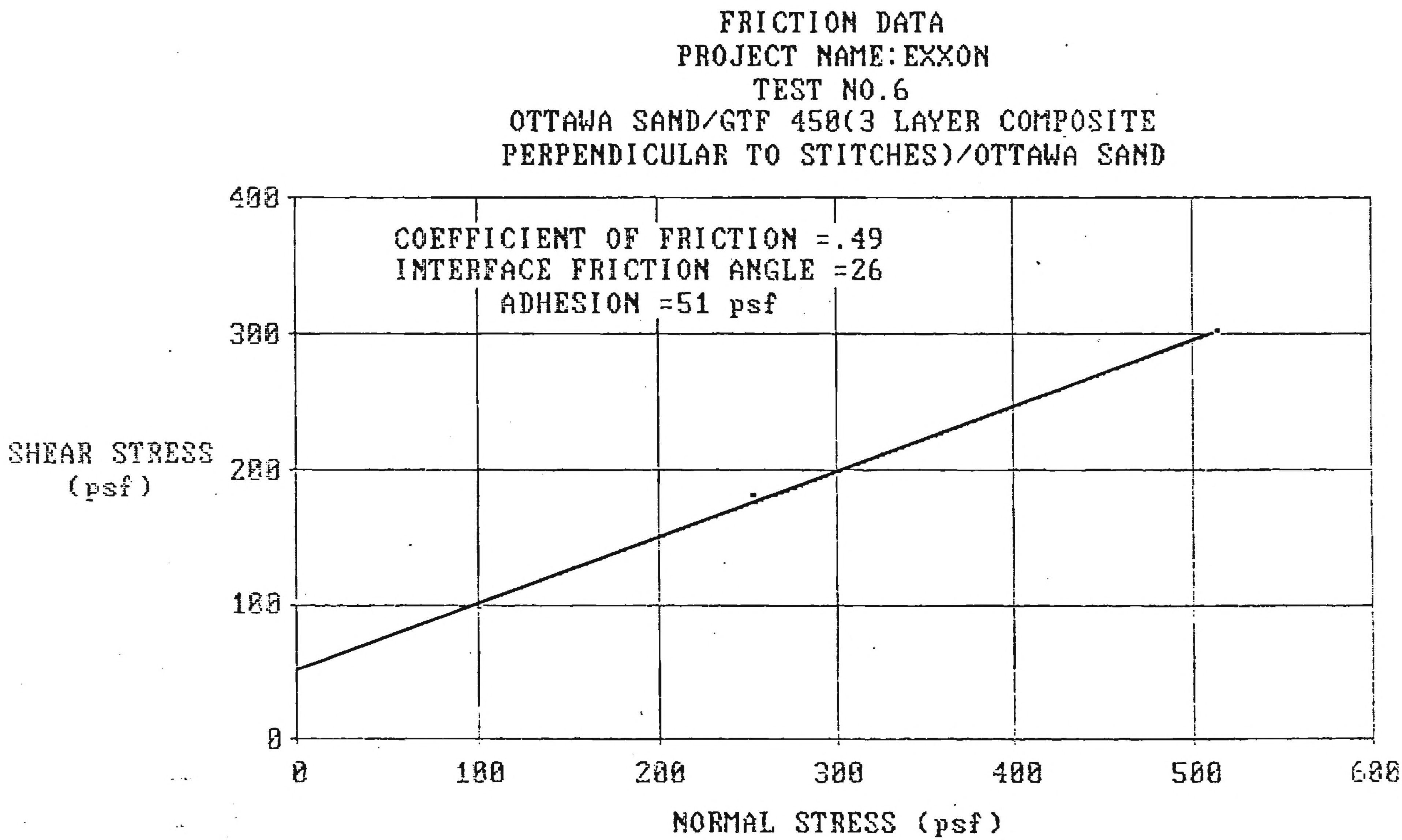

FRICTION DATA
PROJECT NAME:EXXON
TEST NO.5
OTTAWA SAND/GTF 450(3 LAYER COMPOSITE
PARALLEL TO STITCHES)/OTTAWA SAND
COEFFICIENT OF FRICTION =.50
INTERFACE FRICTION ANGLE =26.5
ADHESION =49 psf
SHEAR STRESS (psf)
400
300
200
100
0
0
100
200
300
400
500
600
NORMAL STRESS (psf)

Figure 4-6



FRICTION DATA
PROJECT NAME:EXXON
TEST NO.6
OTTAWA SAND/GTF 450(3 LAYER COMPOSITE
PERPENDICULAR TO STITCHES)/OTTAWA SAND
COEFFICIENT OF FRICTION =.49
INTERFACE FRICTION ANGLE =26
ADHESION =51 psf
SHEAR STRESS
(psf)
400
300
200
100
0
0
100
200
300
400
500
600
NORMAL STRESS (psf)

Figure 4-7

FRICTION DATA
PROJECT NAME: EXXON
TEST NO.1
CLAYEYSAND/GTF 500(PARALLEL STITCHES)/CLAYEY SAND

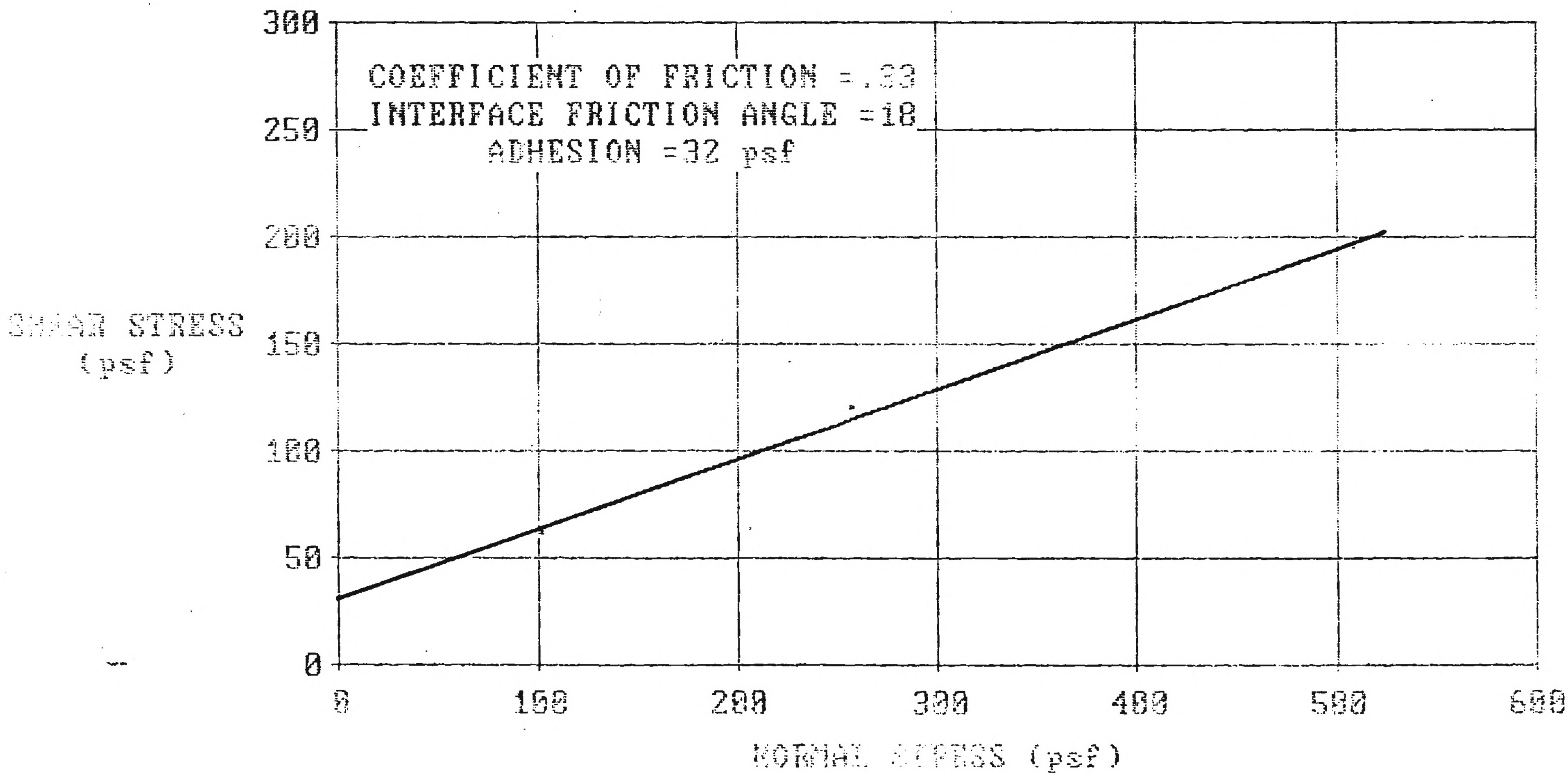


Figure 4-8

FRICTION DATA
PROJECT NAME:EXXON

CLAYEYSAND/GTF 500(PERPENDICULAR TO STITCHES)/CLAYEY SAND

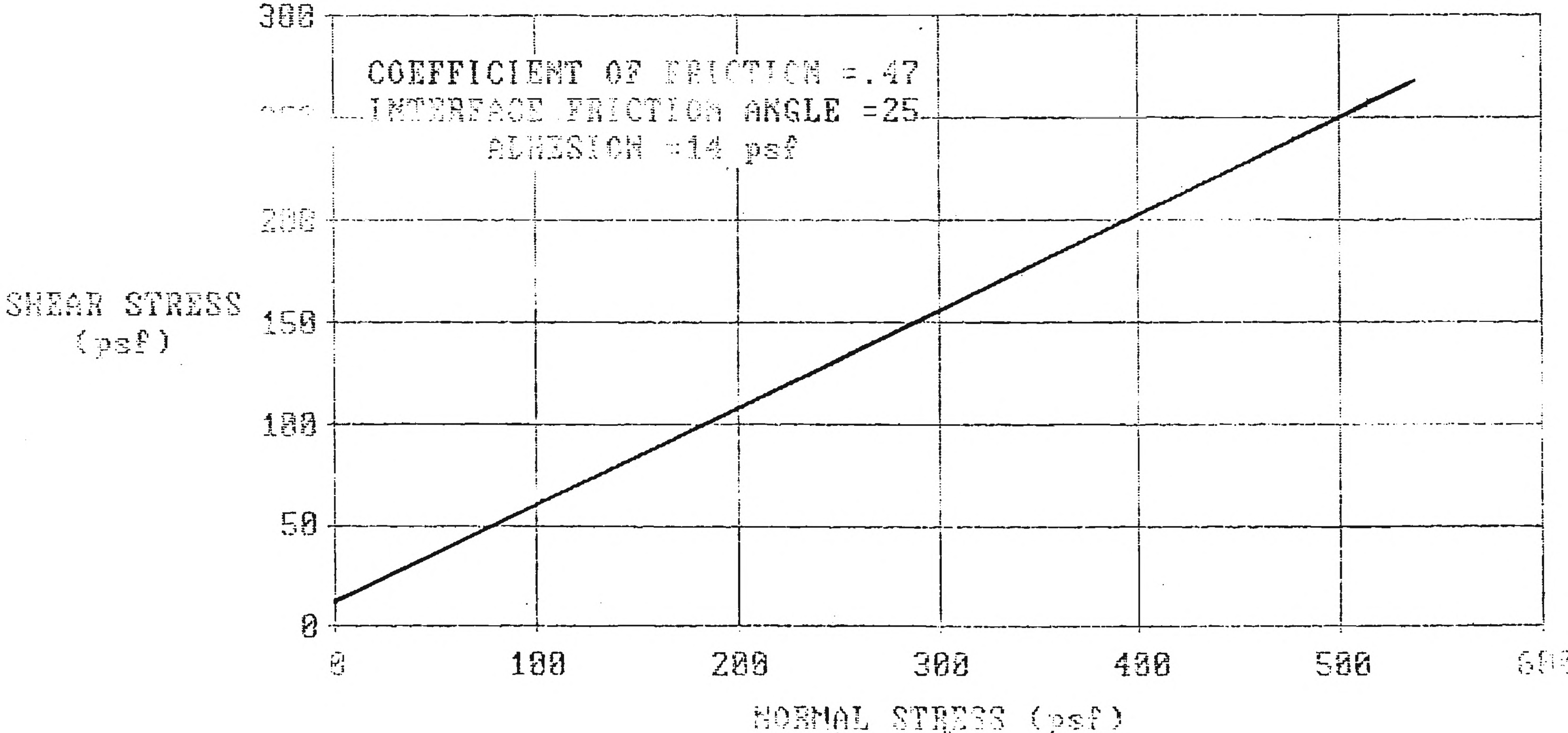


Figure 4-9

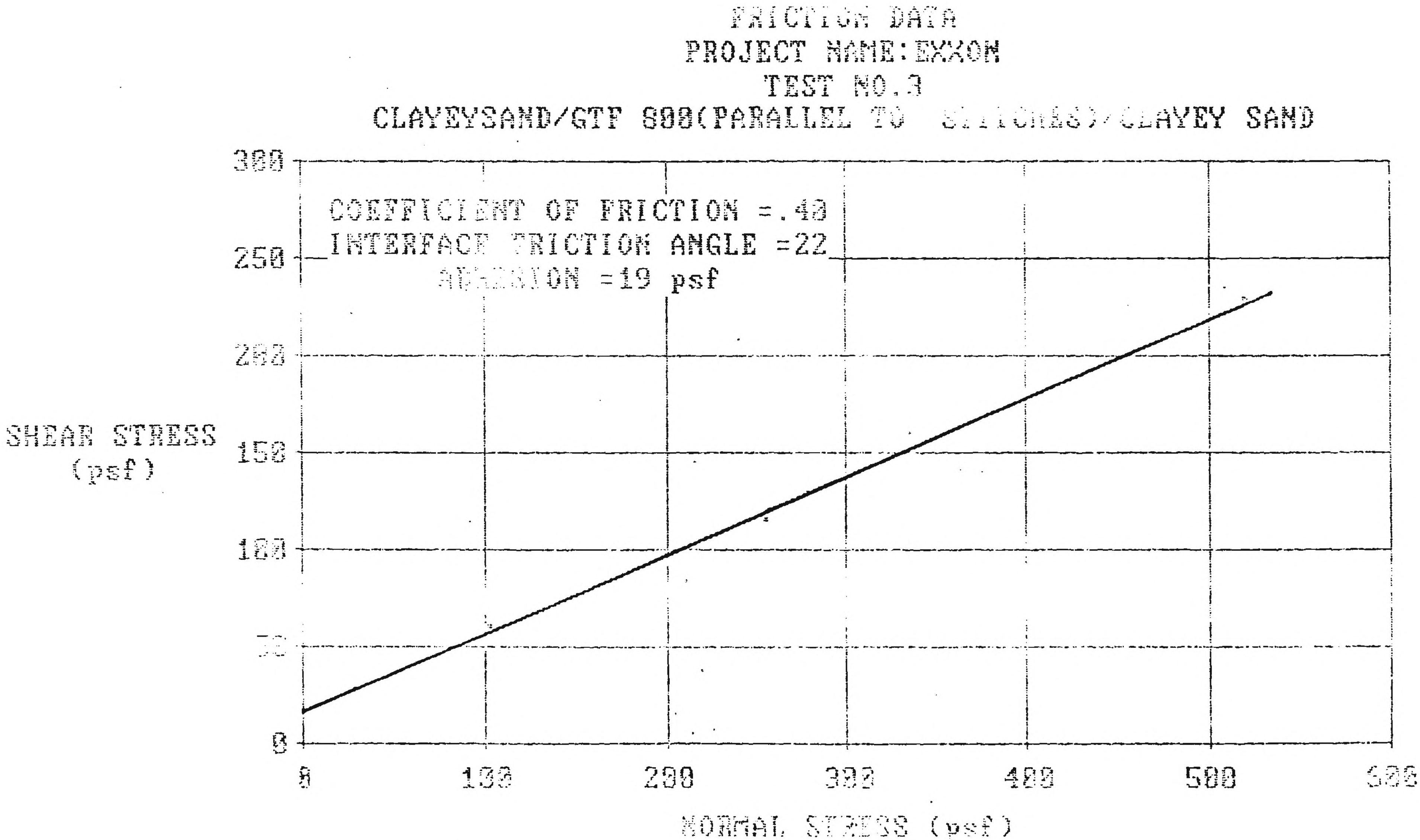

FRICTION DATA
PROJECT NAME:EXXON
TEST NO.3
CLAYEYSAND/GTF 800(PARALLEL TO STITCHES)/CLAYEY SAND
COEFFICIENT OF FRICTION =.40
INTERFACE FRICTION ANGLE =22
ADHESION =19 psf
SHEAR STRESS
(psf)
300
250
200
150
100
50
0
0
100
200
300
400
500
600
NORMAL STRESS (psf)

Figure 4-10

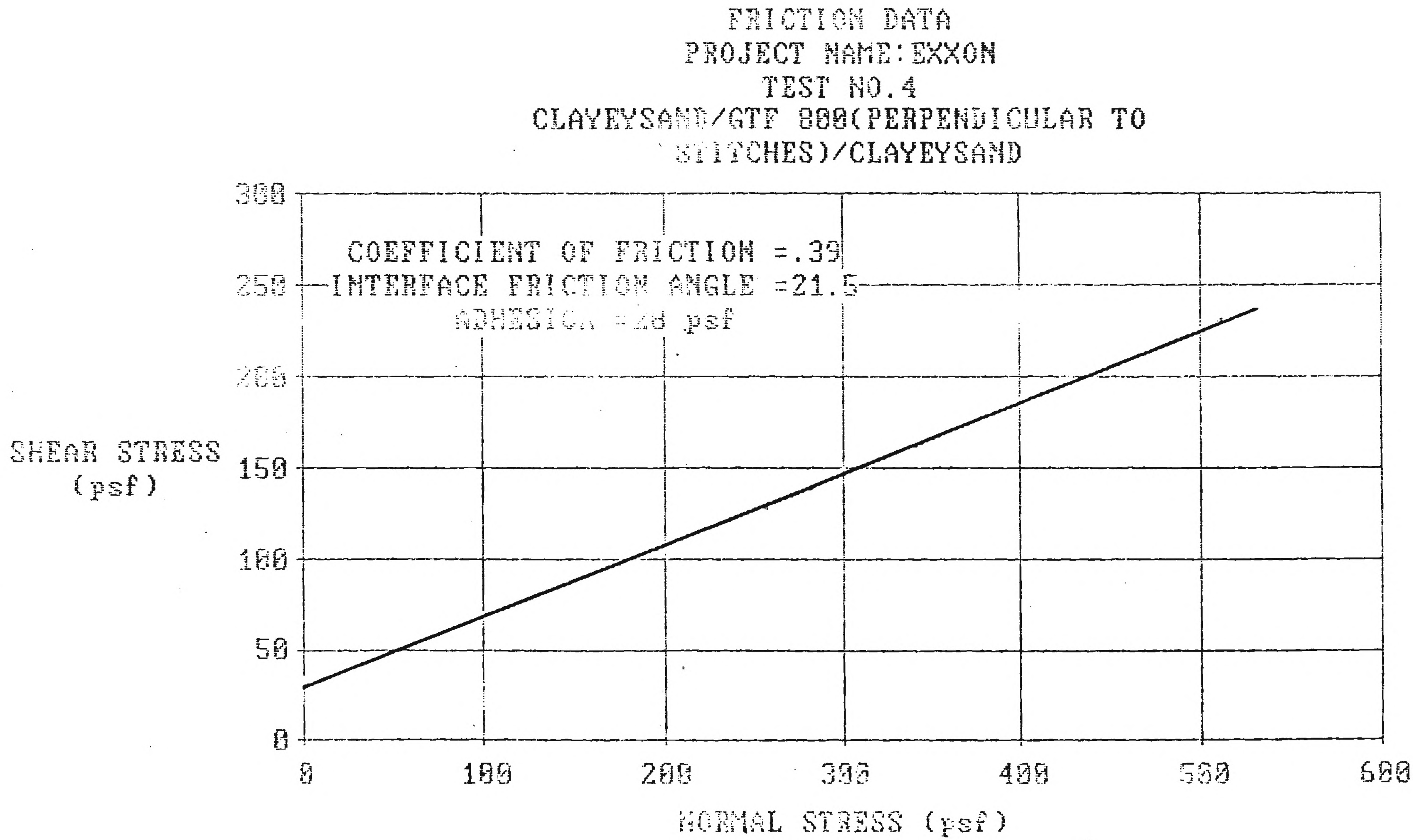

FRICTION DATA
PROJECT NAME:EXXON
TEST NO.4
CLAYEYSAND/GTF 800(PERPENDICULAR TO
STITCHES)/CLAYEYSAND
COEFFICIENT OF FRICTION =.39
INTERFACE FRICTION ANGLE =21.5
ADHESION =28 psf
SHEAR STRESS
(psf)
300
250
200
150
100
50
0
0
100
200
300
400
500
600
NORMAL STRESS (psf)

Figure 4-11

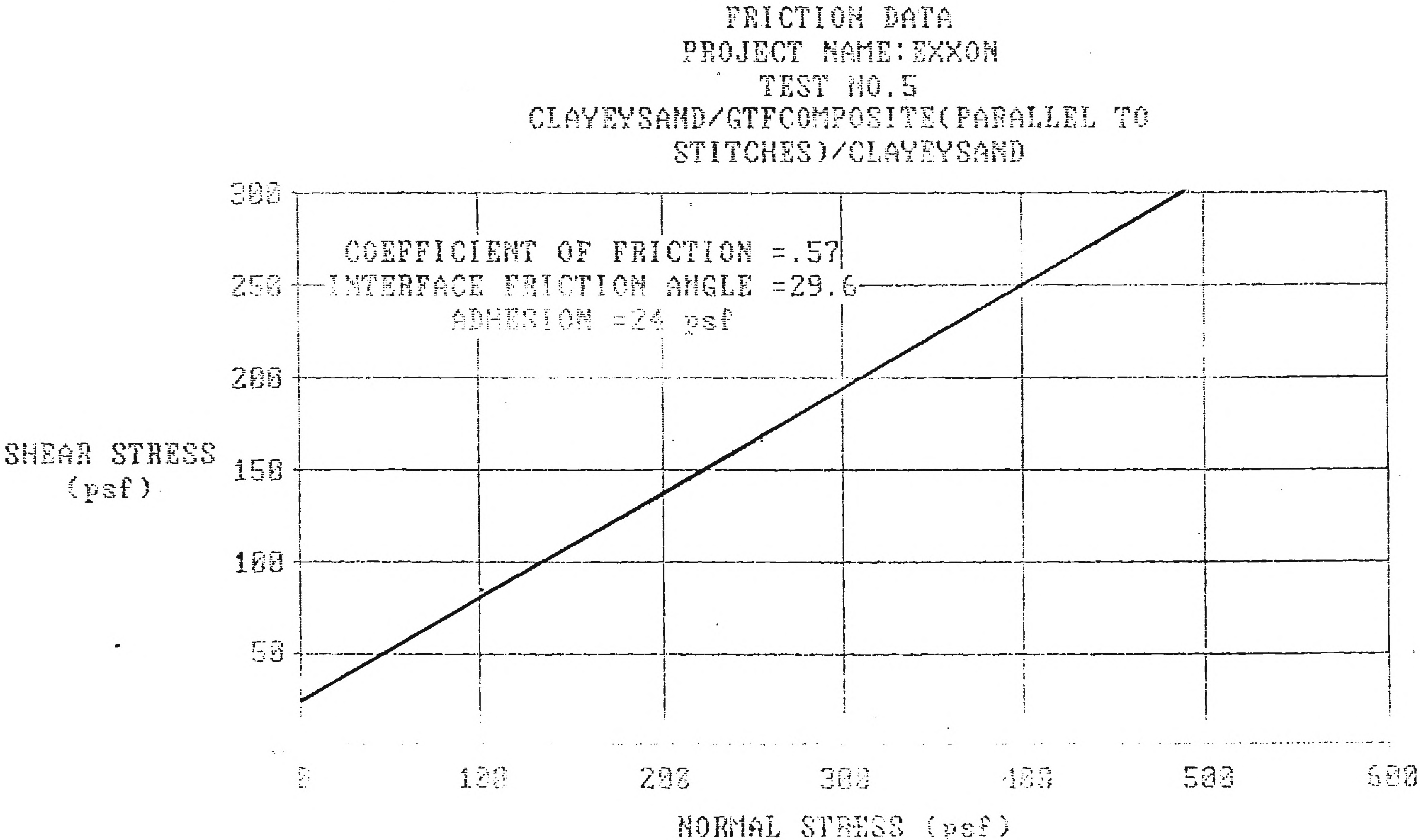

FRICTION DATA
PROJECT NAME:EXXON
TEST NO.5
CLAYEYSAND/GTFCOMPOSITE(PARALLEL TO
STITCHES)/CLAYEYSAND
COEFFICIENT OF FRICTION =.57
INTERFACE FRICTION ANGLE =29.6
ADHESION =24 psf
SHEAR STRESS
(psf)
300
250
200
150
100
50
0
100
200
300
400
500
600
NORMAL STRESS (psf)

Figure 4-12

FRICTION DATA
PROJECT NAME:EXXON
TEST NO.6
CLAYEYSAND/GTFCOMPOSITE(PERPENDICULAR TO STITCHES)/CLAYEYSAND

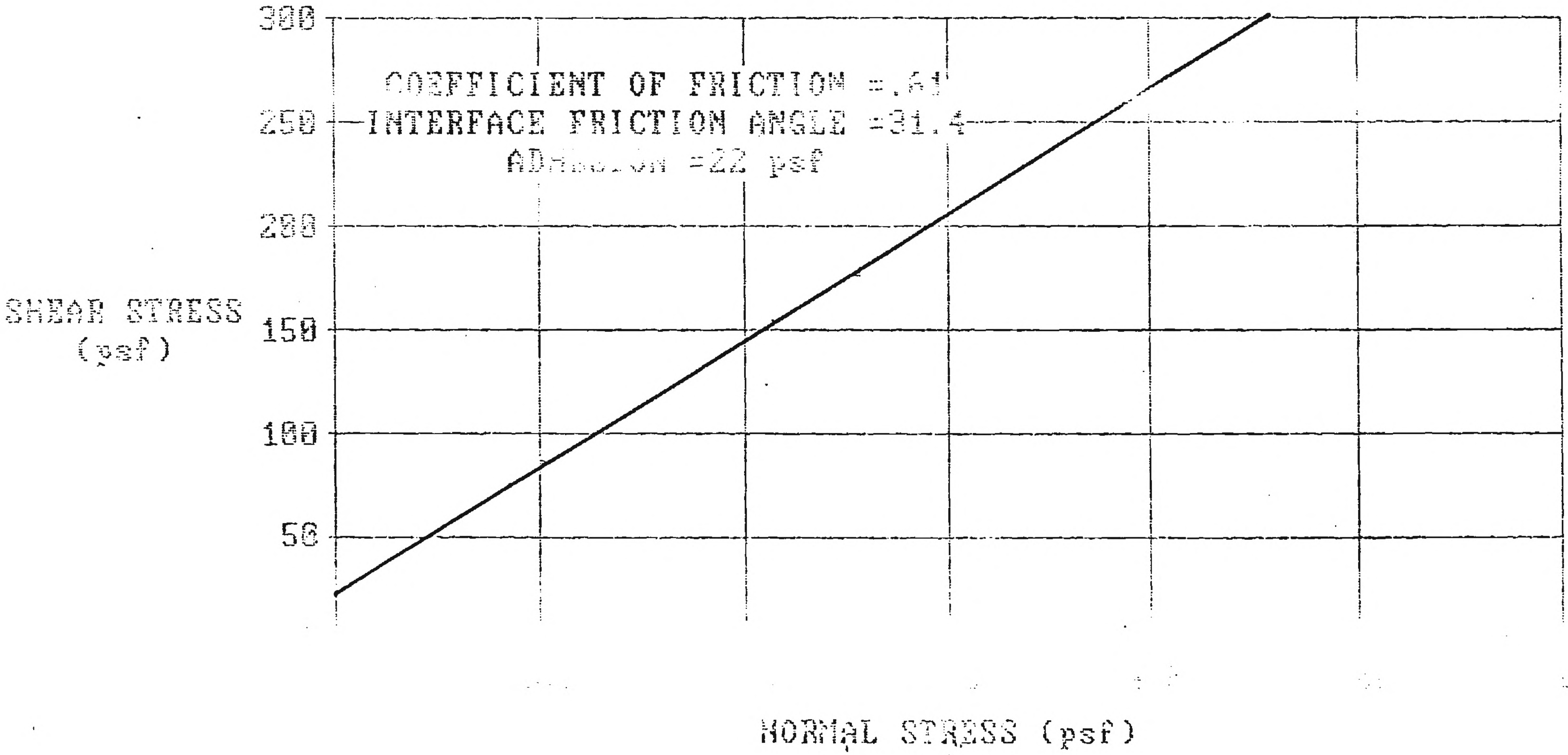
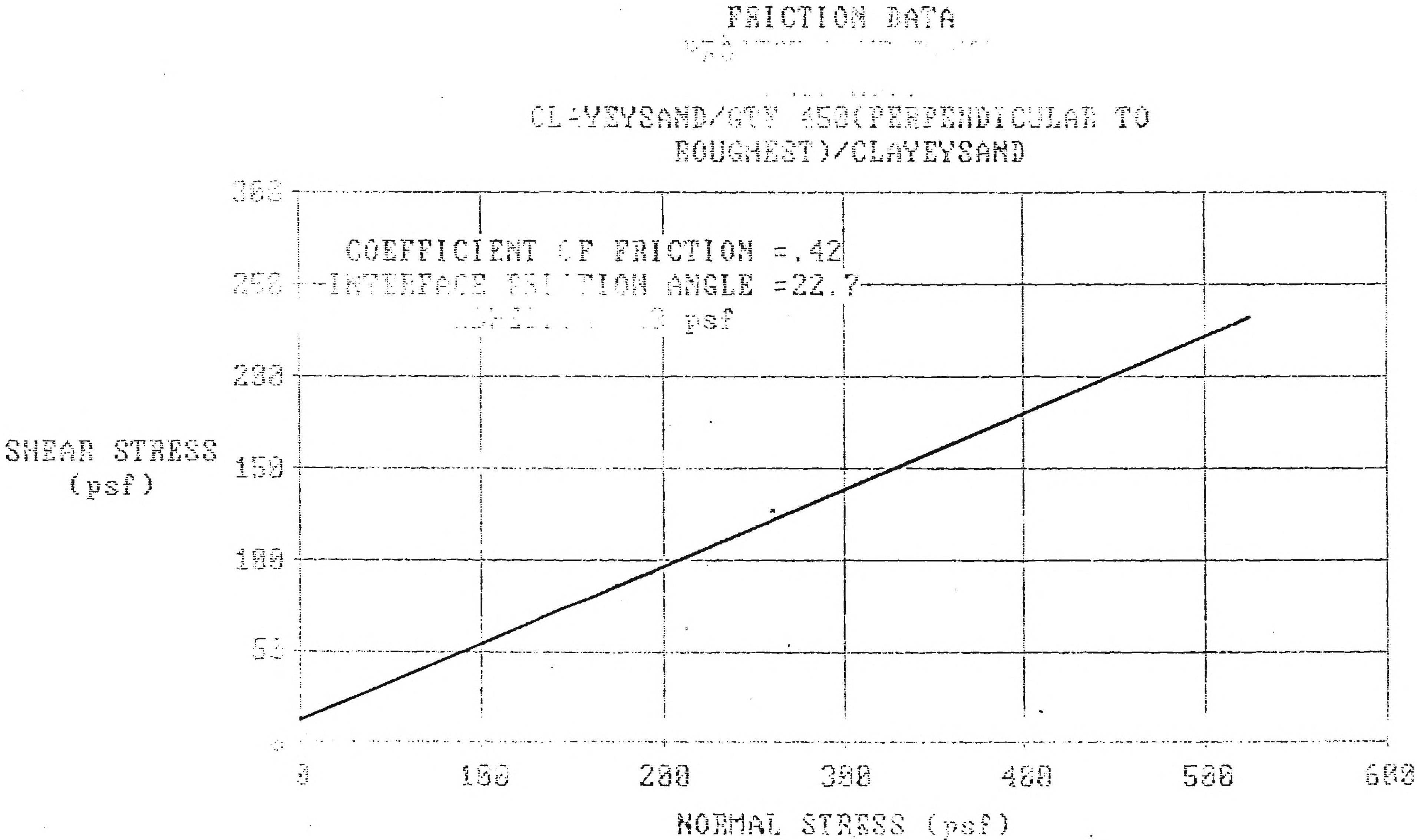


Figure 4-13



FRICTION DATA
CLAYEYSAND/GTF 450(PERPENDICULAR TO
ROUGHEST)/CLAYEYSAND
COEFFICIENT OF FRICTION =.42
INTERFACE FRICTION ANGLE =22.7
psf
SHEAR STRESS
(psf)
300
250
200
150
100
50
0
0
100
200
300
400
500
600
NORMAL STRESS (psf)

Figure 4-14

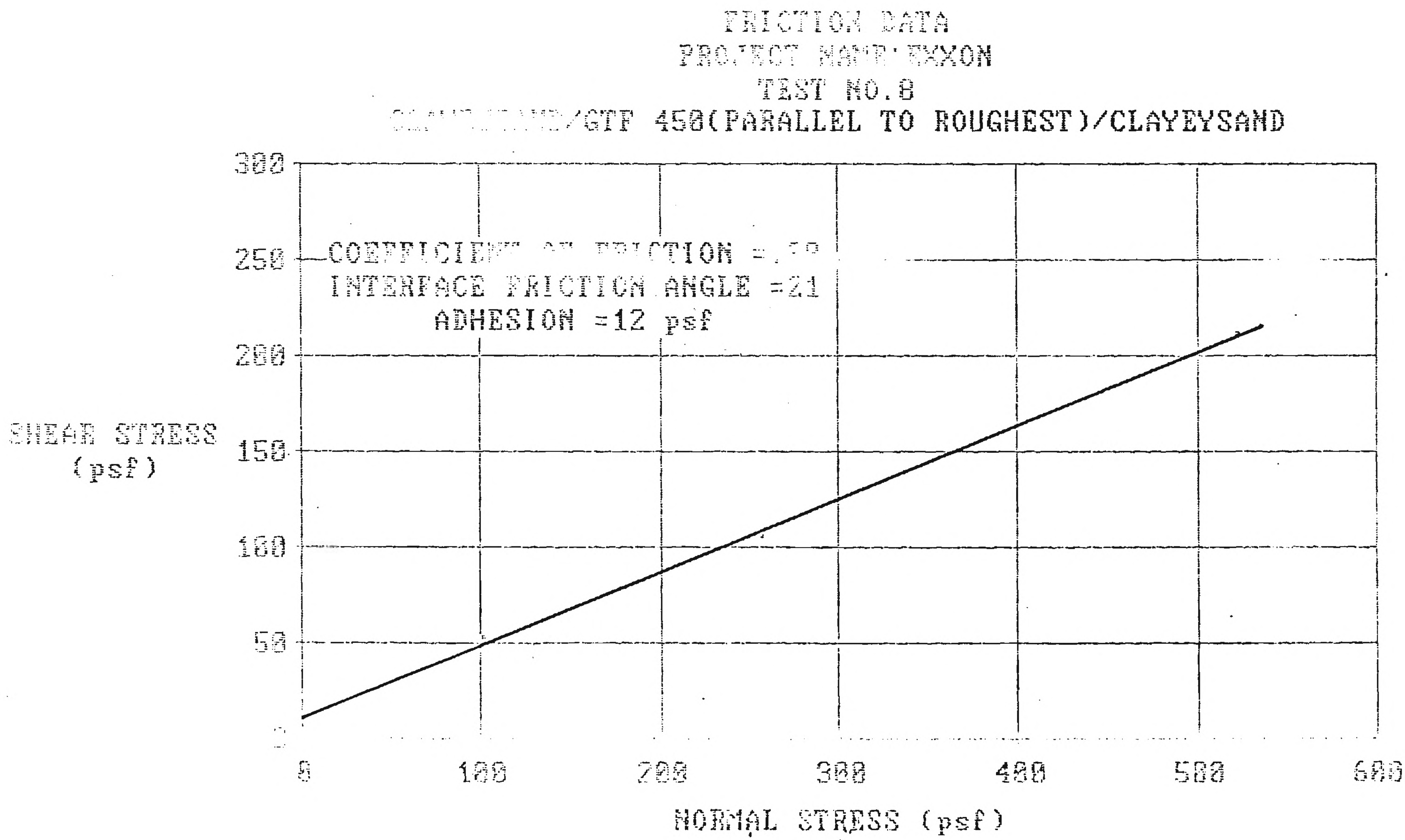
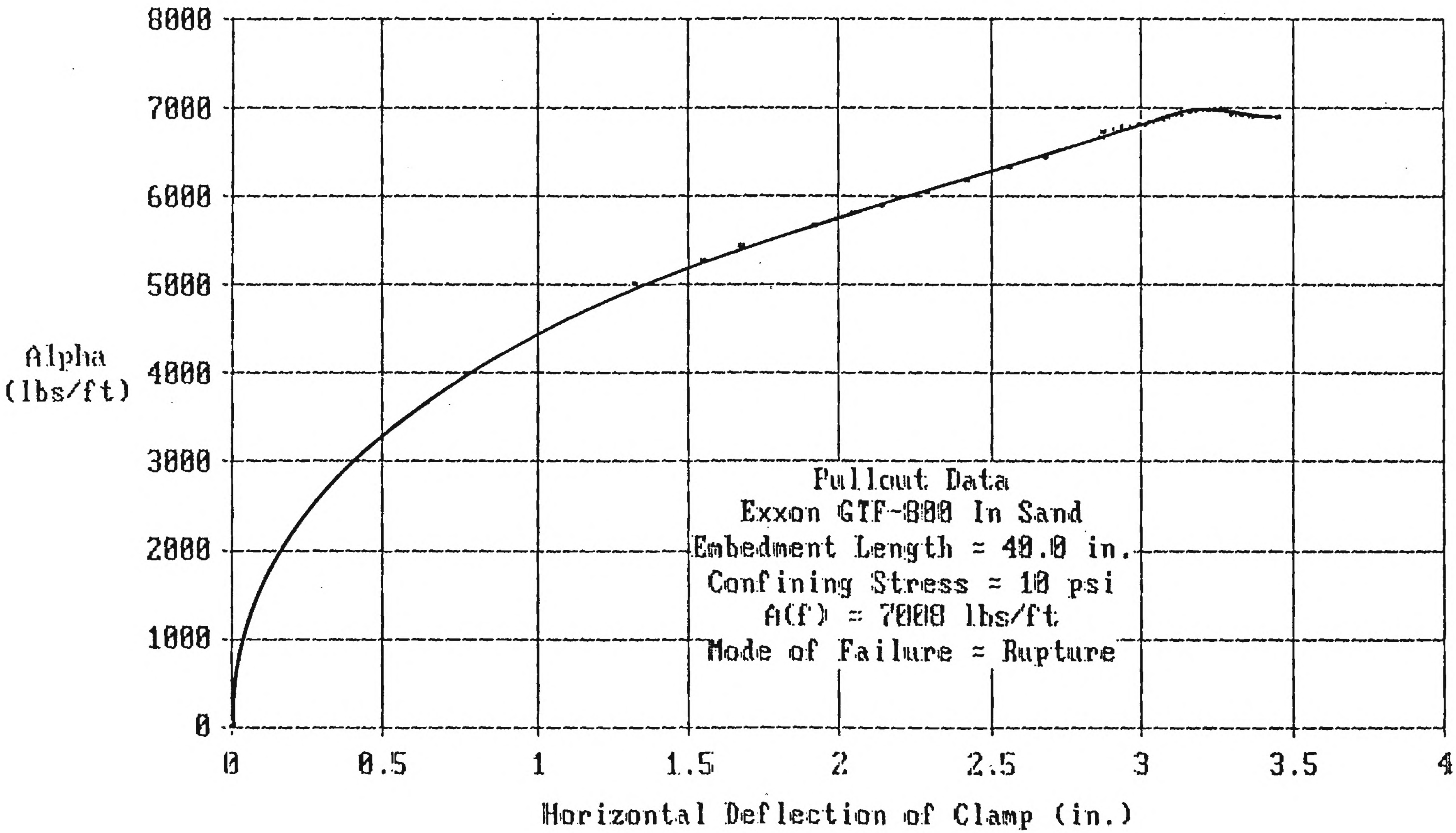

FRICTION DATA
PROJECT NAME:EXXON
TEST NO.8
/GTF 450(PARALLEL TO ROUGHEST)/CLAYEYSAND
INTERFACE FRICTION ANGLE =21
ADHESION =12 psf
SHEAR STRESS (psf)
300
250
200
150
100
50
0
0
100
200
300
400
500
600
NORMAL STRESS (psf)

Figure 4-15

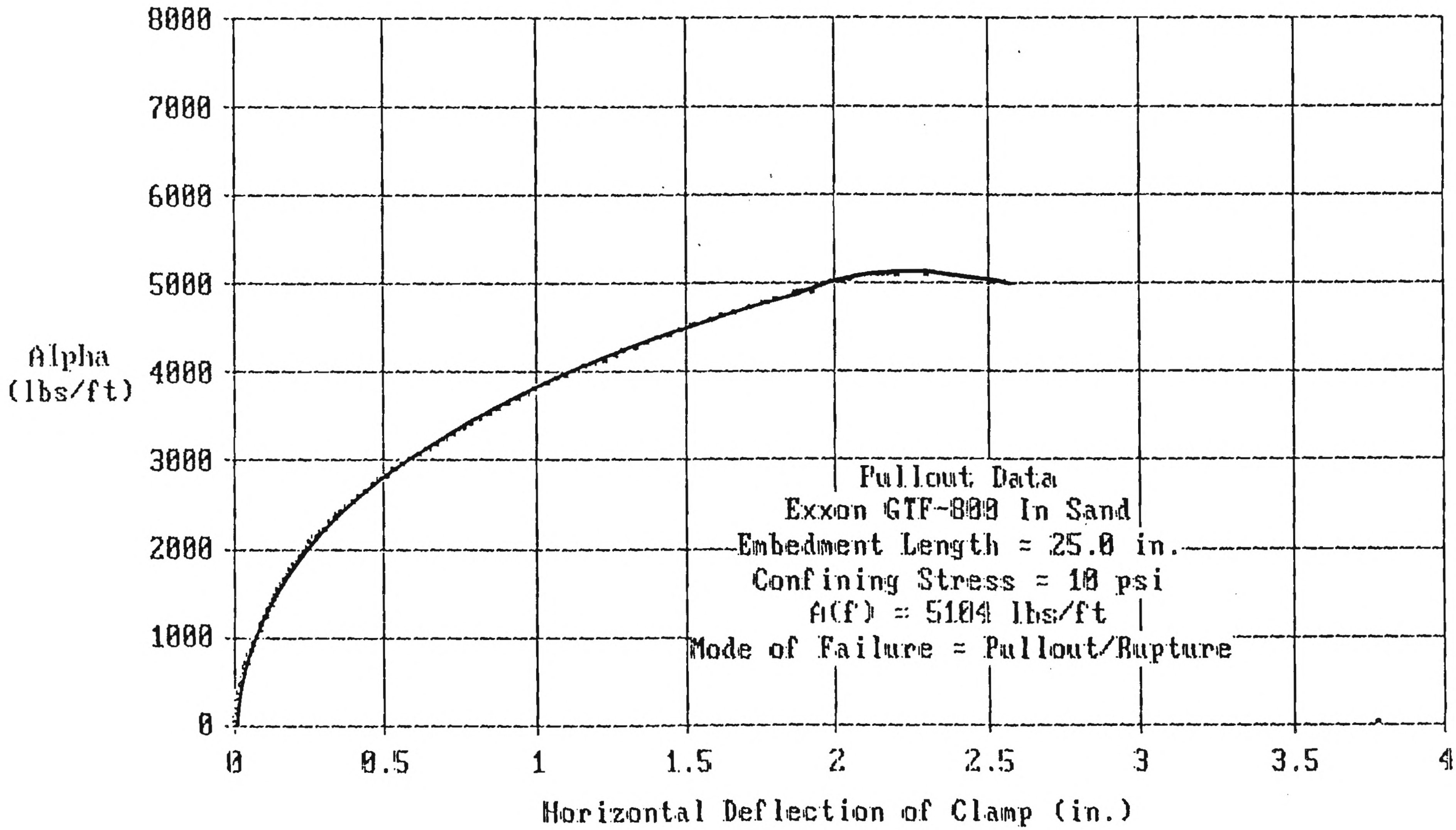
APPENDIX C.

PULLOUT RESULTS



8000
7000
6000
5000
4000
3000
2000
1000
0
Alpha
(lbs/ft)
Pullout Data
Exxon GTF-800 In Sand
Embedment Length = 40.0 in.
Confining Stress = 10 psi
A(f) = 7000 lbs/ft
Mode of Failure = Rupture
0
0.5
1
1.5
2
2.5
3
3.5
4
Horizontal Deflection of Clamp (in.)

Figure (5-20)


8000
7000
6000
5000
4000
3000
2000
1000
0
Alpha
(lbs/ft)
0
0.5
1
1.5
2
2.5
3
3.5
4
Pullout Data
Exxon GTF-800 In Sand
Embedment Length = 25.0 in.
Confining Stress = 10 psi
A(f) = 5104 lbs/ft
Mode of Failure = Pullout/Rupture
Horizontal Deflection of Clamp (in.)
Figure (5-21)

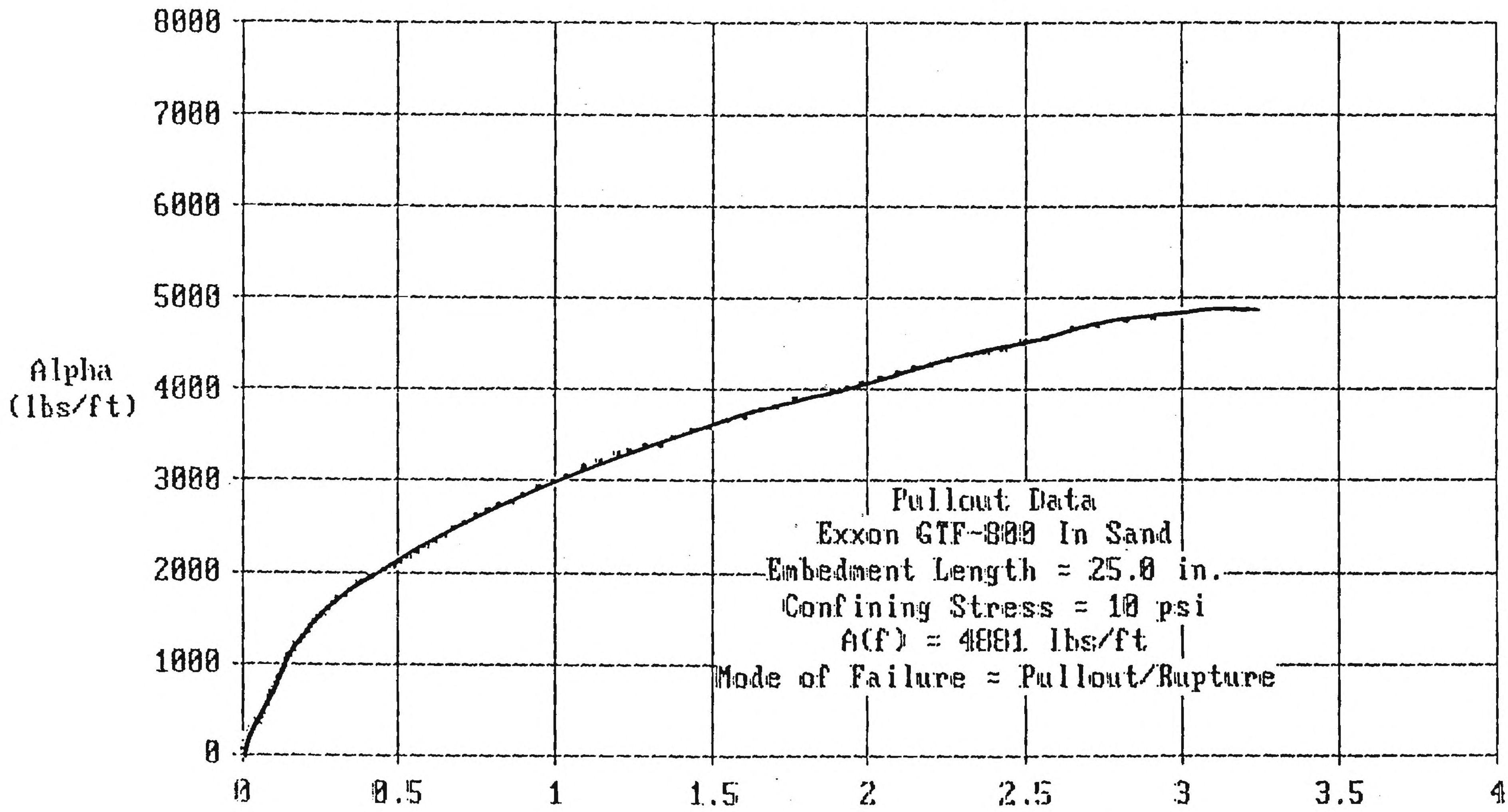
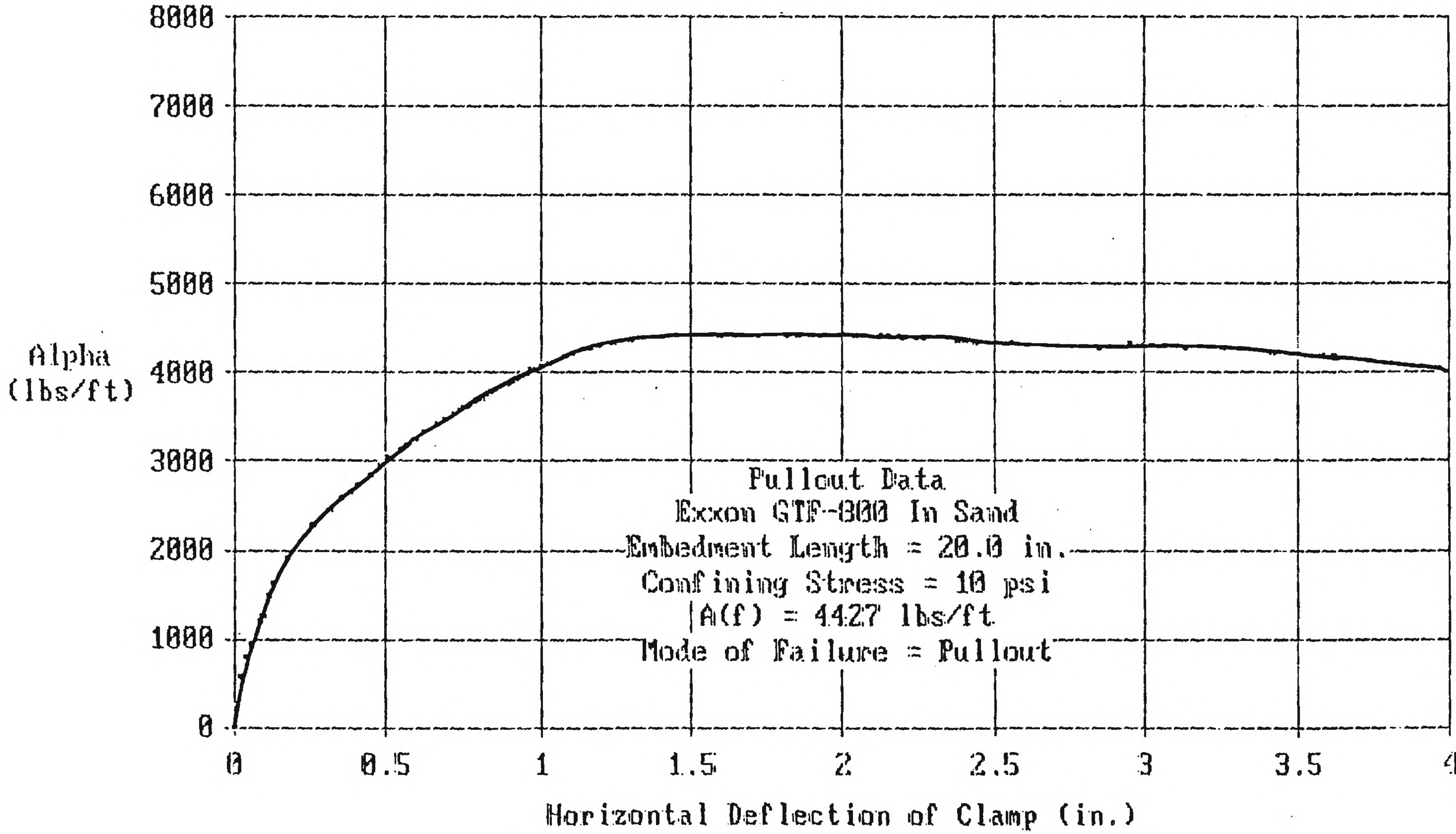

8000
7000
6000
5000
4000
3000
2000
1000
0
Alpha
(lbs/ft)
0
0.5
1
1.5
2
2.5
3
3.5
4
Pullout Data
Exxon GTF-800 In Sand
Embedment Length = 25.0 in.
Confining Stress = 10 psi
A(f) = 4881 lbs/ft
Mode of Failure = Pullout/Rupture
Horizontal Deflection of Clamp (in.)

Figure (5-22)



8000
7000
6000
5000
4000
3000
2000
1000
0
Alpha (lbs/ft)
0
0.5
1
1.5
2
2.5
3
3.5
4
Pullout Data
Exxon GTF-800 In Sand
Embedment Length = 20.0 in.
Confining Stress = 10 psi
A(f) = 4427 lbs/ft
Mode of Failure = Pullout
Horizontal Deflection of Clamp (in.)

Figure (5-23)

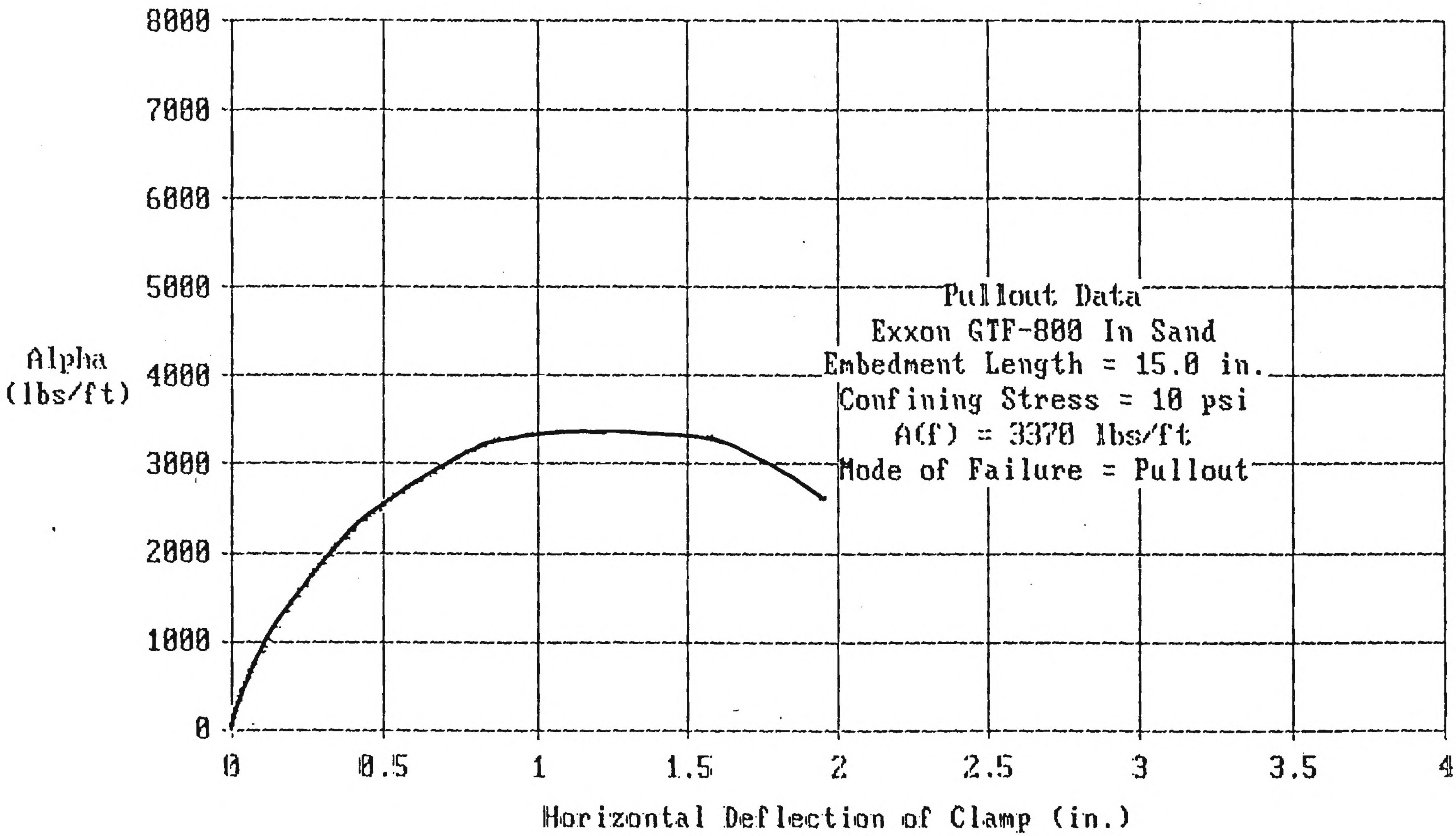
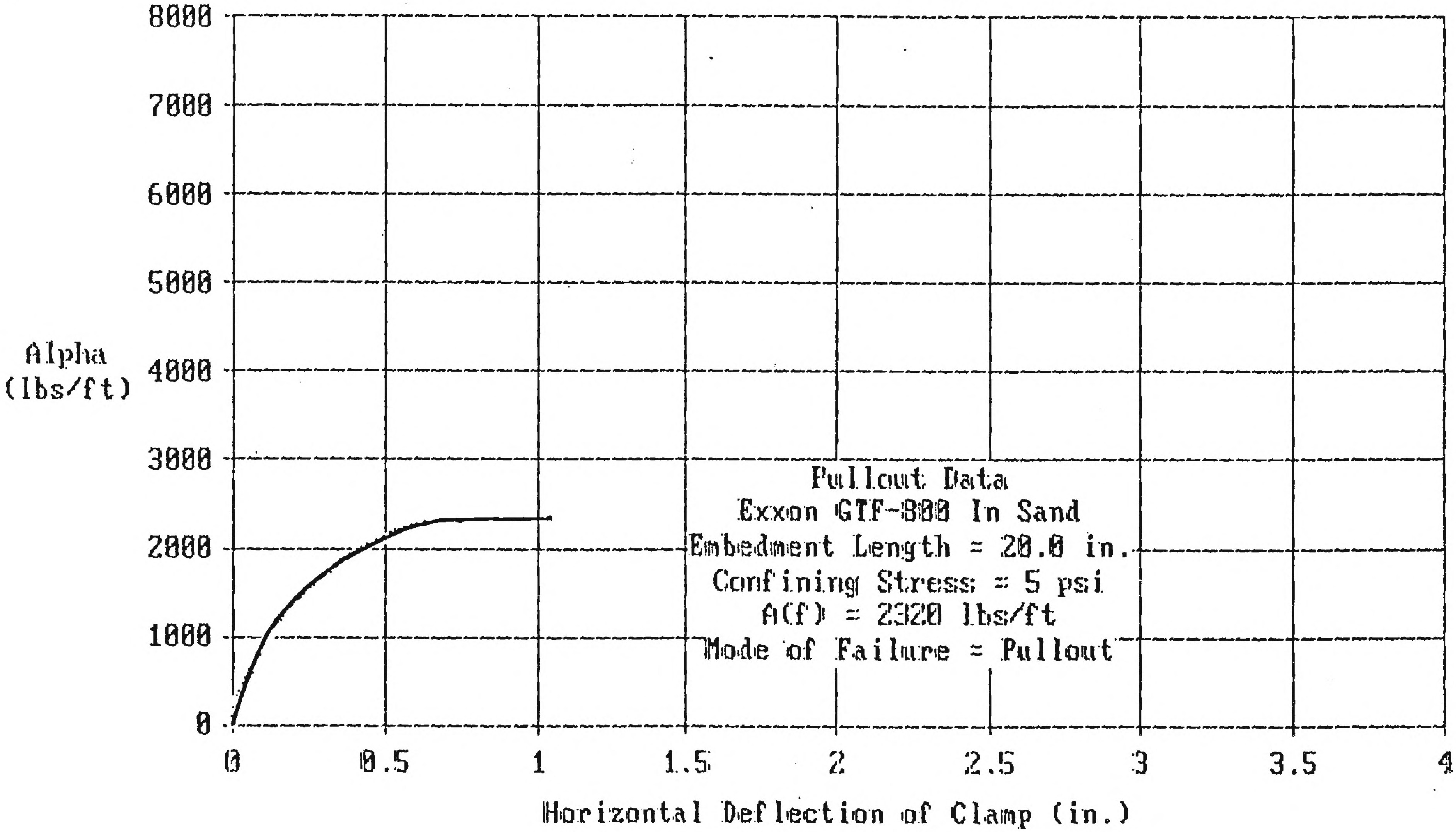
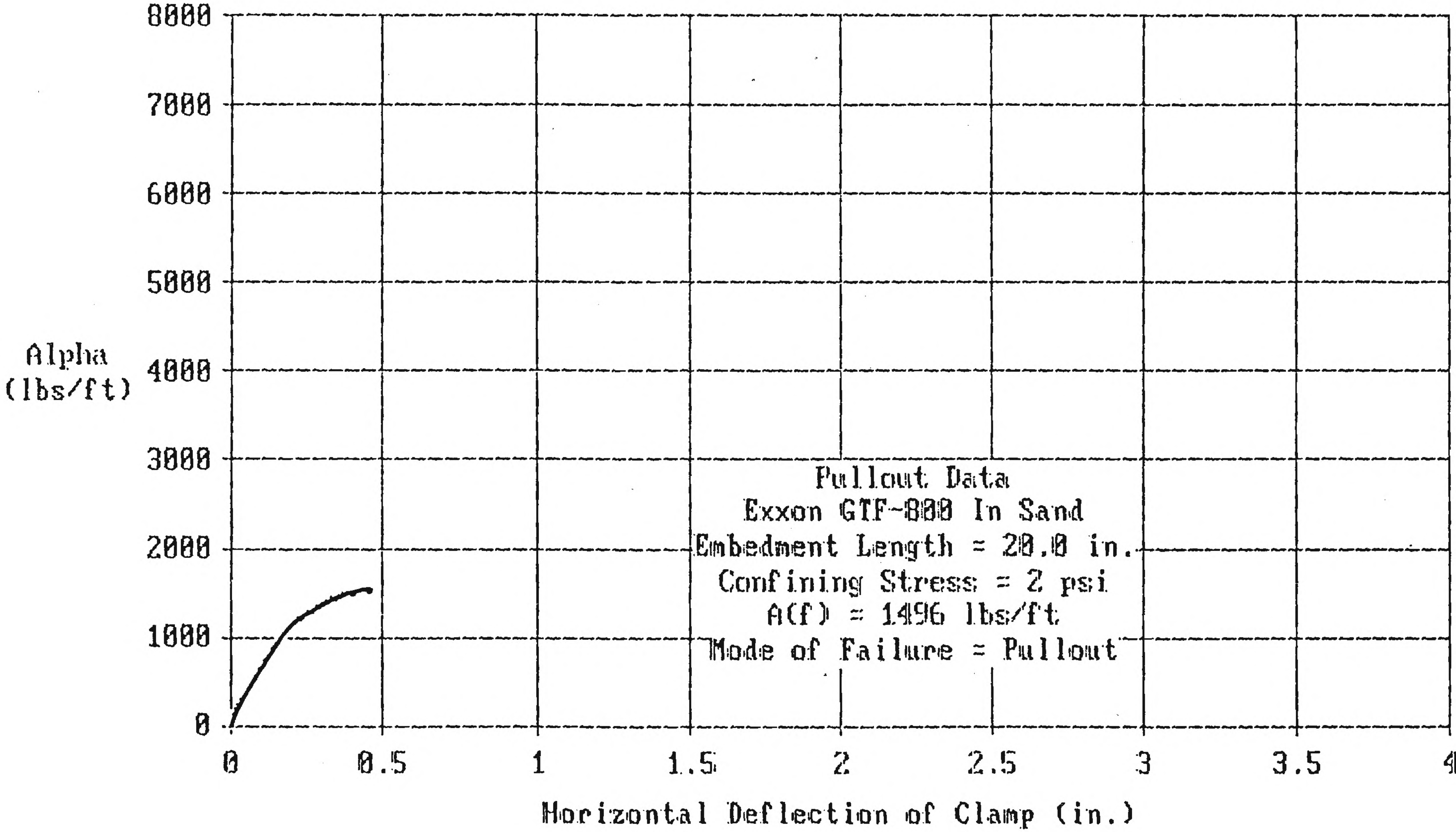

8000
7000
6000
5000
4000
3000
2000
1000
0
Alpha
(lbs/ft)
0
0.5
1
1.5
2
2.5
3
3.5
4
Horizontal Deflection of Clamp (in.)
Pullout Data
Exxon GTF-800 In Sand
Embedment Length = 15.0 in.
Confining Stress = 10 psi
A(f) = 3370 lbs/ft
Mode of Failure = Pullout

Figure (5-24)


8000
7000
6000
5000
4000
3000
2000
1000
0
Alpha
(lbs/ft)
0
0.5
1
1.5
2
2.5
3
3.5
4
Pullout Data
Exxon GTF-800 In Sand
Embedment Length = 20.0 in.
Confining Stress = 5 psi
A(f) = 2320 lbs/ft
Mode of Failure = Pullout
Horizontal Deflection of Clamp (in.)
Figure (5-25)



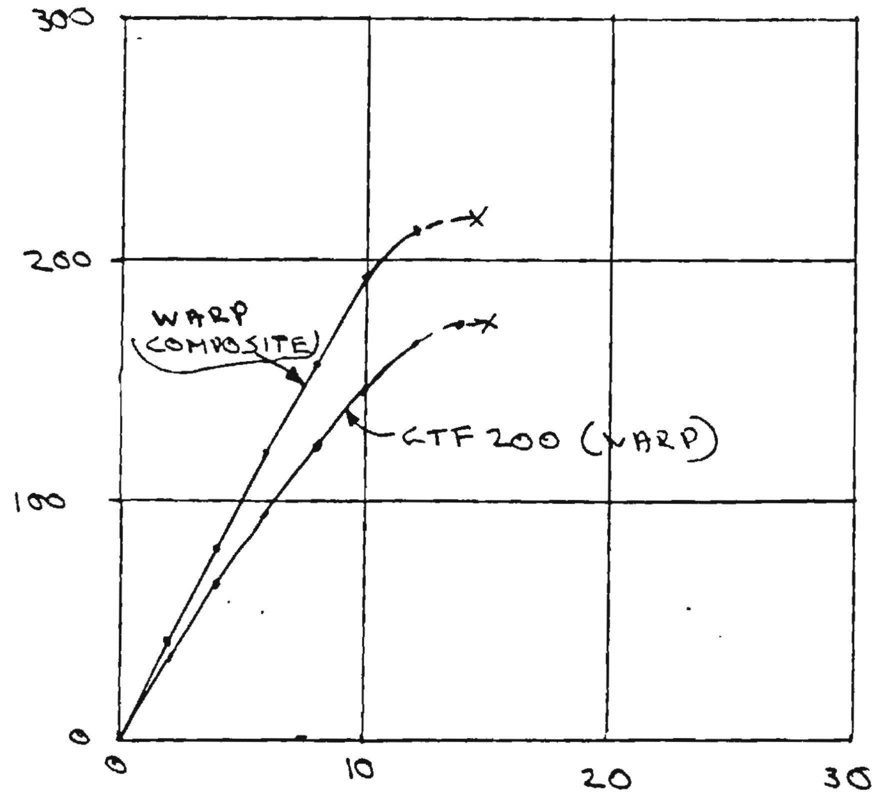
8000
7000
6000
5000
4000
3000
2000
1000
0
Alpha
(lbs/ft)
0
0.5
1
1.5
2
2.5
3
3.5
4
Pullout Data
Exxon GTF-800 In Sand
Embedment Length = 20.0 in.
Confining Stress = 2 psi
A(f) = 1496 lbs/ft
Mode of Failure = Pullout
Horizontal Deflection of Clamp (in.)

Figure (5-26)

APPENDIX D.

MANUFACTURER INDEX TEST RESULTS

GTF 150D/ GTF 200 / GTF 130 D



WIDE WIDTH RESULTS WOVEN/ NONWOVEN COMPOSITE

OCT 2, 1986

Exxon GeoTextiles

Technical Specification Data

EXXON CHEMICAL COMPANY

CUSTOM ENGINEERED FABRIC

EXXON CHEMICALS

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(404) 955-2300
Telex 794-588-EXX CHEM AMER

Exxon GeoTextiles

Technical Specification Data

GEOTEXTILE PRODUCT

TECHNICAL SPECIFICATION DATA

Properties	Test Method	GTF-350
Grab Tensile Strength (lbs.)	ASTM D-1682/.01.85.02 (Proposed)	300 X 350
Wide Width Strip Tensile (lbs./in.)	ASTM D-4595	240 X 280
Secant Modulus @ 10% Elongation (lbs./in.)	ASTM D-4595	1000 X 2000
Elongation (%)	ASTM D-1682/.01.85.02 (Proposed)	20
Trapezoid Tear (lbs.)	ASTM D-1117/.01.80.01 (Proposed)	170 X 170
Puncture (lbs.)	ASTM D-751/.01.82.03 (Proposed)	185
Mullen Burst (psi)	ASTM D-751/3786	600+
Permeability Coefficient-k (cm/sec)	ASTM D-4491-85 Falling Head	.01
Ultra Violet Stability (Strength Retained %)	FEDTM-191 500 Hrs. Exposure	>70
Apparent Opening Size CW02215 (U.S. Sieve No. Equivalent)	ASTM .03.81.08 (Proposed)	80/40
Thickness (mils)	-	-
Weight (oz/SY)	ASTM 4491	7
Vertical Water Flow Rate (GPM/SF)	Falling Head (8 to 2 cm)	35

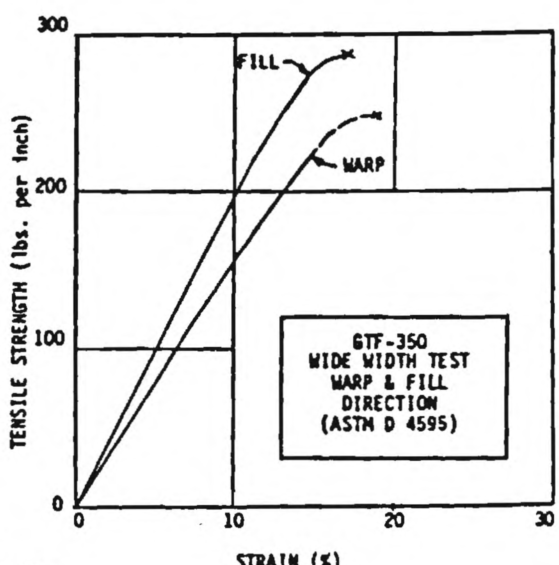
Primary Application — Soil Separation and Reinforcement

The above test results are minimum average roll properties.

Packaging

Roll Width (ft.)	12.5	15.0
Roll Weight (lbs.)	270	270
Roll Length (ft.)	426	354
Roll Area (SY)	600	600

10/86



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GEOTEXTILE PRODUCT

TECHNICAL SPECIFICATION DATA

Properties	Test Method	GTF-450
Grab Tensile Strength (lbs.)	ASTM D-1682/.01.85.02 (Proposed)	400
Wide Width Strip Tensile (lbs./in.)	ASTM D-4595	270
Secant Modulus @ 10% Elongation (lbs./in.)	ASTM D-4595	2000
Elongation (%)	ASTM D-1682/.01.85.02 (Proposed)	20
Trapezoid Tear (lbs.)	ASTM D-1117/.01.80.01 (Proposed)	190
Puncture (lbs.)	ASTM D-751/.01.82.03 (Proposed)	220
Mullen Burst (psi)	ASTM D-751/3786	600+
Permeability Coefficient-k (cm/sec)	ASTM D-4491-85 Falling Head	.01
Ultra Violet Stability (Strength Retained %)	FEDTM-191 500 Hrs. Exposure	>70
Apparent Opening Size CW02215 (U.S. Sieve No. Equivalent)	ASTM .03.81.08 (Proposed)	70/50
Thickness (mils)	-	-
Weight (oz/SY)	ASTM 4491	8
Vertical Water Flow Rate (GPM/SF)	Falling Head (8 to 2 cm)	50

Primary Application Soil Separation and Reinforcement

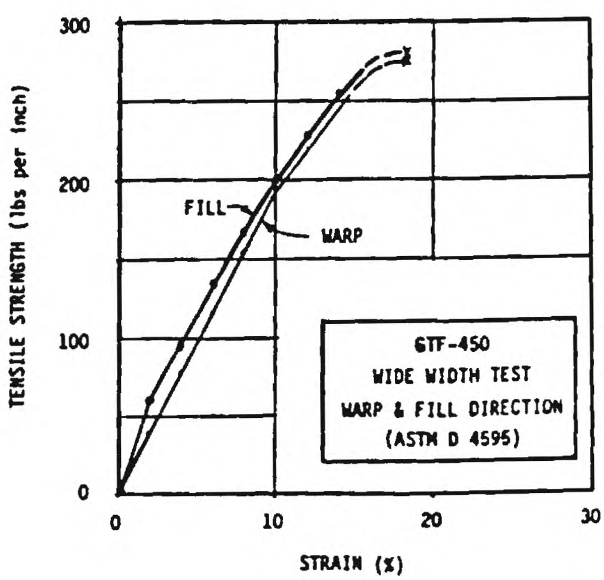
The above test results are minimum average roll properties.

Packaging

Roll Width (ft.)	12.5	15.0
Roll Weight (lbs.)	250	250
Roll Length (ft.)	360	300
Roll Area (SY)	500	500

*Silt fence products produced from GTF-450 fabric are designated GTF-455.

10/86



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GEOTEXTILE PRODUCT

TECHNICAL SPECIFICATION DATA

Properties	Test Method	GTF-500
Grab Tensile Strength (lbs.)	ASTM D-1682/.01.85.02 (Proposed)	750 x 620
Wide Width Strip Tensile (lbs./in.)	ASTM D-4595	440 x 400
Secant Modulus @ 10% Elongation (lbs./in.)	ASTM D-4595	3500
Elongation (%)	ASTM D-1682/.01.85.02 (Proposed)	20
Trapezoid Tear (lbs.)	ASTM D-1117/.01.80.01 (Proposed)	200 x 200
Puncture (lbs.)	ASTM D-751/.01.82.03 (Proposed)	400
Mullen Burst (psi)	ASTM D-751/3786	1500+
Permeability Coefficient-k (cm/sec)	ASTM D-4491-85 Falling Head	.01
Ultra Violet Stability (Strength Retained %)	FEDTM-191 500 Hrs. Exposure	>70
Apparent Opening Size CW02215 (U.S. Sieve No. Equivalent)	ASTM .03.81.08 (Proposed)	100+
Thickness (mills)	-	55
Weight (oz/SY)	ASTM 4491	13.5
Vertical Water Flow Rate (GPM/SF)	Falling Head (8 to 2 cm)	14

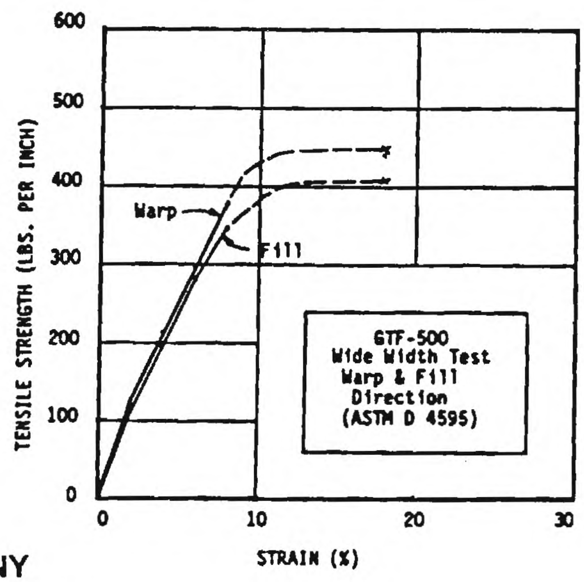
Primary Application Soil Separation and Reinforcement

The above test results are minimum average roll properties.

Packaging

Roll Width (ft.)	12.5
Roll Weight (lbs.)	275
Roll Length (ft.)	150
Roll Area (SY)	209

10/86



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Technical Specification Data

GEOTEXTILE PRODUCT

TECHNICAL SPECIFICATION DATA

Properties	Test Method	GTF-650
Grab Tensile Strength (lbs.)	ASTM D-1682/.01.85.02 (Proposed)	1150 x 650
Wide Width Strip Tensile (lbs./in.)	ASTM D-4595	800 x 475
Secant Modulus @ 10% Elongation (lbs./in.)	ASTM D-4595	5000 x 4000
Elongation (%)	ASTM D-1682/.01.85.02 (Proposed)	20
Trapezoid Tear (lbs.)	ASTM D-1117/.01.80.01 (Proposed)	550 x 200
Puncture (lbs.)	ASTM D-751/.01.82.03 (Proposed)	550
Mullen Burst (psi)	ASTM D-751/3786	1500
Permeability Coefficient-k (cm/sec)	ASTM D-4491-85 Falling Head	.01
Ultra Violet Stability (Strength Retained %)	FEDTM-191 500 Hrs. Exposure	>70
Apparent Opening Size CW02215 (U.S. Sieve No. Equivalent)	ASTM .03.81.08 (Proposed)	100+
Thickness (mills)	-	62
Weight (oz/SY)	ASTM 4491	21
Vertical Water Flow Rate (GPM/SF)	Falling Head (8 to 2 cm)	14

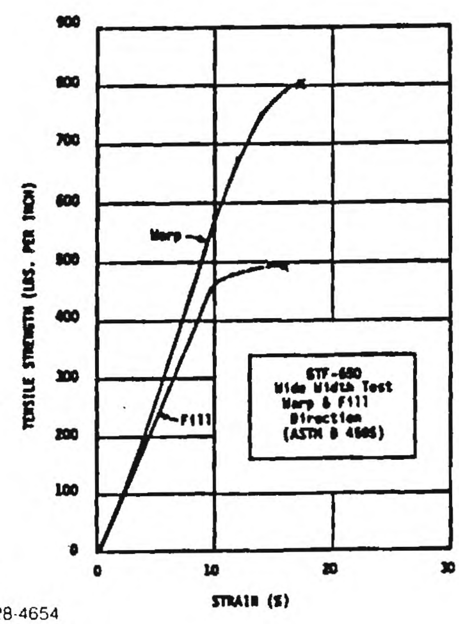
Primary Application — Soil Separation and Reinforcement

The above test results are minimum average roll properties.

Packaging

Roll Width (ft.)	12.5
Roll Weight (lbs.)	230
Roll Length (ft.)	150
Roll Area (SY)	208

10/86



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Technical Specification Data

GEOTEXTILE PRODUCT

TECHNICAL SPECIFICATION DATA

Properties	Test Method	GTF-800
Grab Tensile Strength (lbs.)	ASTM D-1682/.01.85.02 (Proposed)	900 x 825
Wide Width Strip Tensile (lbs./in.)	ASTM D-4595	590 x 530
Secant Modulus @ 10% Elongation (lbs./in.)	ASTM D-4595	5000
Elongation (%)	ASTM D-1682/.01.85.02 (Proposed)	20
Trapezoid Tear (lbs.)	ASTM D-1117/.01.80.01 (Proposed)	220 x 220
Puncture (lbs.)	ASTM D-751/.01.82.03 (Proposed)	450
Mullen Burst (psi)	ASTM D-751/3786	1500+
Permeability Coefficient-k (cm/sec)	ASTM D-4491-85 Falling Head	.01
Ultra Violet Stability (Strength Retained %)	FEDTM-191 500 Hrs. Exposure	>70
Apparent Opening Size CW02215 (U.S. Sieve No. Equivelent)	ASTM .03.81.08 (Proposed)	100+
Thickness (mills)	-	75
Weight (oz/SY)	ASTM 4491	20
Vertical Water Flow Rate (GPM/SF)	Falling Head (8 to 2 cm)	14

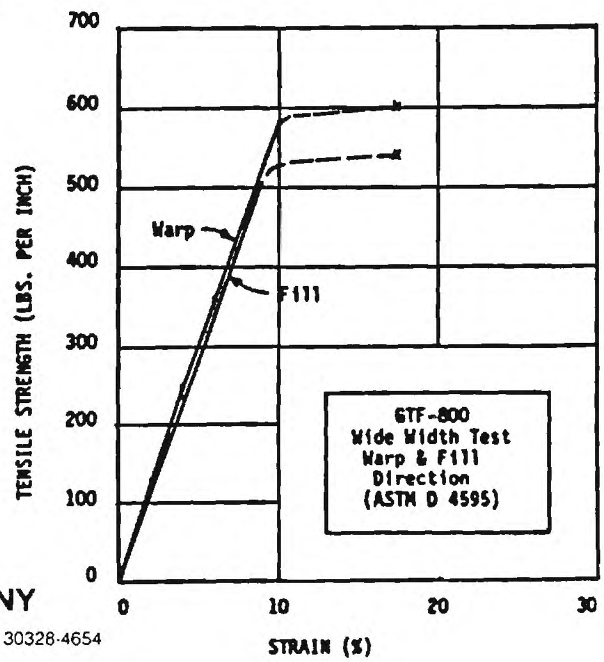
Primary Application — Soil Separation and Reinforcement

The above test results are minimum average roll properties.

Packaging

Roll Width (ft.)	12.5
Roll Weight (lbs.)	275
Roll Length (ft.)	150
Roll Area (SY)	209

10/86



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Technical Specification Data

GEOTEXTILE PRODUCT

TECHNICAL SPECIFICATION DATA

Properties	Test Method	GTF-1000
Grab Tensile Strength (lbs.)	ASTM D-1682/.01.85.02 (Proposed)	1000 x 1350
Wide Width Strip Tensile (lbs./in.)	ASTM D-4595	650 x 980
Secant Modulus @ 10% Elongation (lbs./in.)	ASTM D-4595	5000 x 9000
Elongation (%)	ASTM D-1682/.01.85.02 (Proposed)	20/15
Trapezoid Tear (lbs.)	ASTM D-1117/.01.80.01 (Proposed)	200 x 200
Puncture (lbs.)	ASTM D-751/.01.82.03 (Proposed)	400
Mullen Burst (psi)	ASTM D-751/3786	1500+
Permeability Coefficient-k (cm/sec)	ASTM D-4491-85 Falling Head	.01
Ultra Violet Stability (Strength Retained %)	FEDTM-191 500 Hrs. Exposure	>70
Apparent Opening Size CW02215 (U.S. Sieve No. Equivalent)	ASTM .03.81.08 (Proposed)	100+
Thickness (mills)	-	55
Weight (oz/SY)	ASTM 4491	19
Vertical Water Flow Rate (GPM/SF)	Falling Head (8 to 2 cm)	20
Polymer Composition		PP/PET

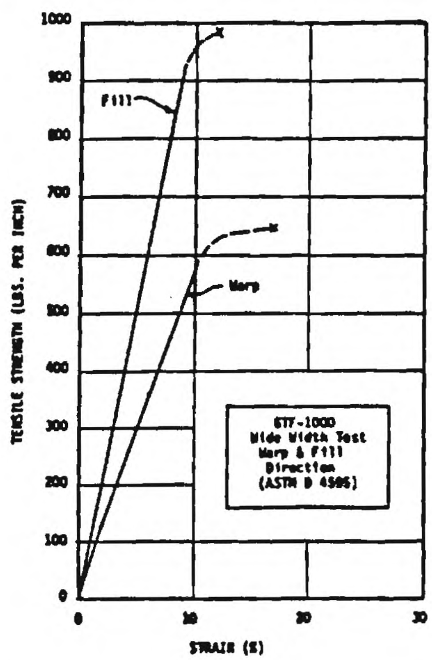
Primary Application Soil Separation and Reinforcement

The above test results are minimum average roll properties.

Packaging

Roll Width (ft.)	12.5
Roll Weight (lbs.)	260
Roll Length (ft.)	150
Roll Area (SY)	209

10/86



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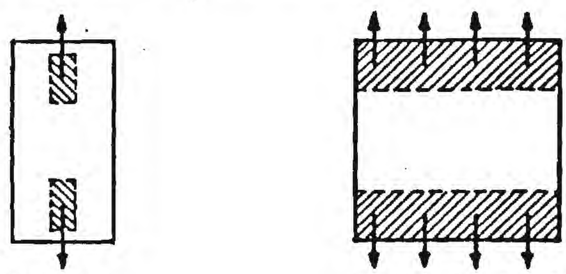
Technical Specification Data

Wide Width Strip Tensile Testing

Wide-Width Strip Tensile Testing
The recent approval of an ASTM standard on wide-width geotextile tensile testing (ASTM D-4595) marks a significant inroad into the acceptance of geotextiles as engineering and construction materials. This test most accurately models the true field stresses that a geotextile will experience during its service life in a reinforcing application.

This technical note is being written to explain the wide-width test, provide understanding of the meaning of test results, and provide background into how an engineer uses these results in his design.

Comparison of Grab Tensile Test With Wide-Width Strip Testing
The sketch below shows a typical test sample subjected to a grab tensile test (ASTM D-1682). This is an index test, used to compare one geotextile to the next. Results are measured in lbs., that being the force required to fail the fabric when tested in this configuration. This result has no relationship whatever to the fabric strength when buried in the ground.



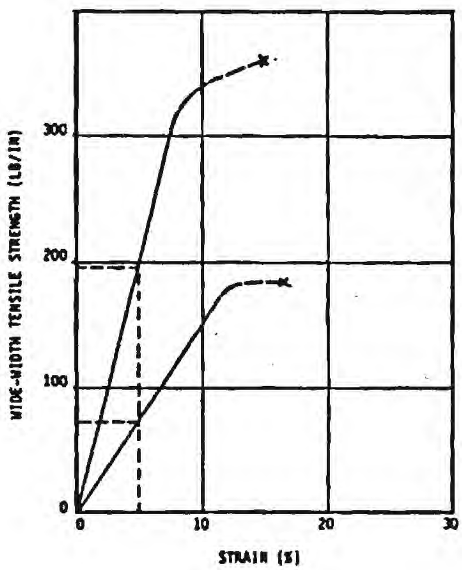
GRAB TENSILE   WIDE-WIDTH STRIP TENSILE

For comparison a wide-width test configuration is also shown indicating the test jaws and the sample as tested. This test is designed to measure the strength of a geotextile when subjected to uniform, full width tensile forces. A geotextile used as internal reinforcement of an earth structure is subjected to very similar stresses. Test results are reported in lbs. per inch width of fabric, as required to tear the geotextile. It is obvious that the tests and the respective results are very different.

Interpretation of Test Results
This plot shows tensile strength versus strain results from wide-width testing. Results are similar to stress strain curve measurements routinely performed on other construction materials, i.e., steel, high strength plastic or concrete. It is an indication of the material's ultimate strength and modulus of elasticity.

Modulus of elasticity or Young's modulus is a material property which is input into classic stress-strain formula and equations. It is a direct indicator of a material deformation or strain under load. A high modulus geotextile develops strength under small strains. A low modulus geotextile must experience large deformations before strength becomes significant.

Results of two geotextile tests have been reported, one a lighter grade geotextile, the other a high strength one. The difference in results is dramatic. The high strength geotextile develops a much higher ultimate tensile strength, while exhibiting much greater elastic properties; higher strength at the same strain. The importance of this will be discussed in the next section.



GeoTextile Design Parameters
In the design of a reinforced geotextile embankment, the strain induced in the soil must be comparable and consistent with the strain induced in the geotextile. This strain compatibility is vital to the interaction of the two materials. A geotextile that requires high elongation before significant strength is obtained, may not be of reinforcing benefit in that once the strain required to stress the geotextile is reached the embankment may have already failed. This strain compatibility can be evaluated by comparing the soil modulus and the modulus of the geotextile at the required strain. It is not uncommon for very soft soils to experience strains of 5 or 10% before failure is reached. Conversely high stiffness soils may see peak strengths develop at 2 and 3% strain. The geotextile strength input to the design must be consistent with that anticipated in the soil structure.

A Word About Exxon Geotextiles
Exxon Custom Engineered Fabrics are constructed for high tensile strength and high elastic modulus requirements in mind. Specification sheets provide wide-width tensile test results and load strain curves. With this data the designer can specify the appropriate geotextile based on both ultimate tensile strength and elastic modulus. In this manner, compatibility between the soil and the geotextile can be assured.

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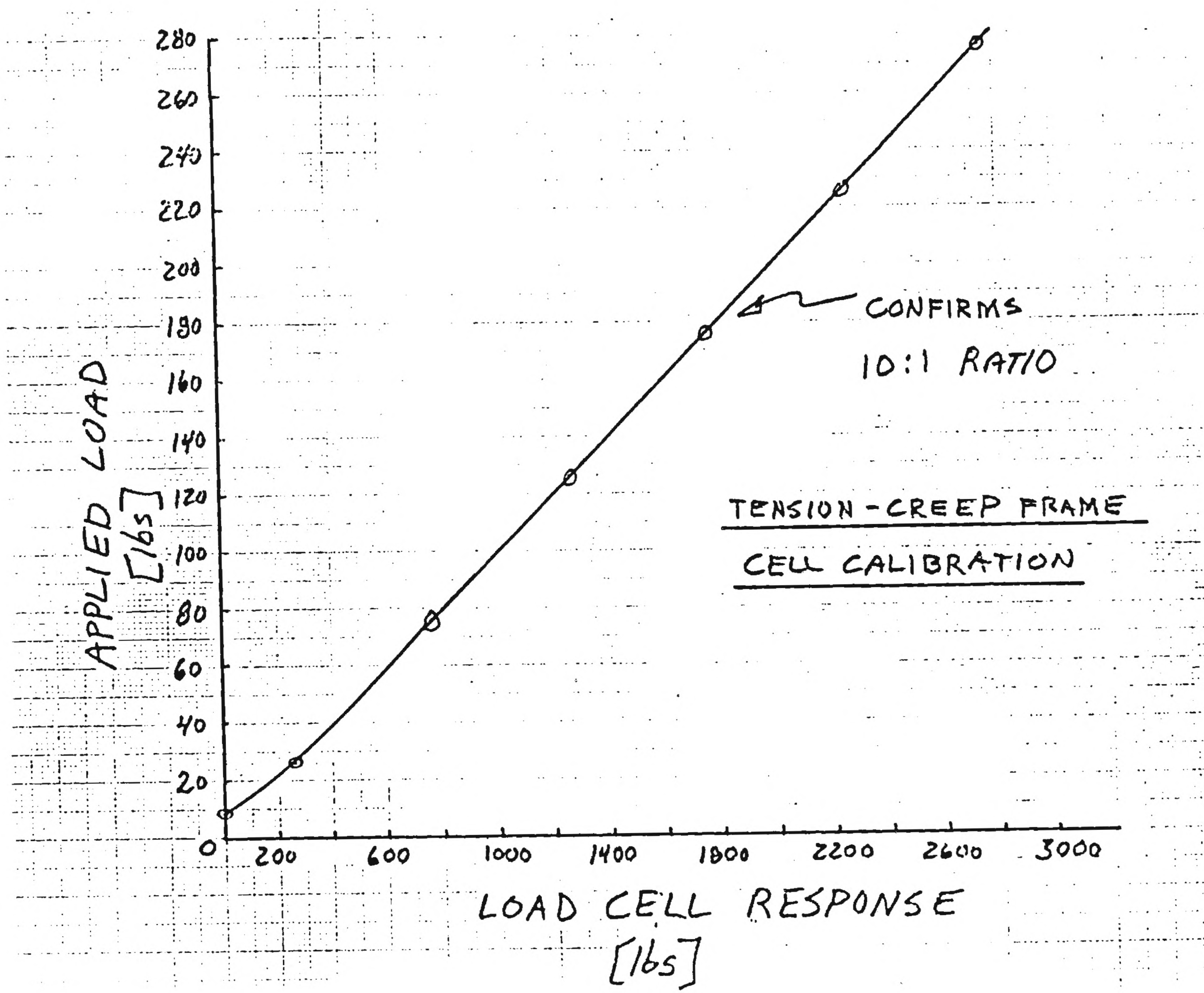
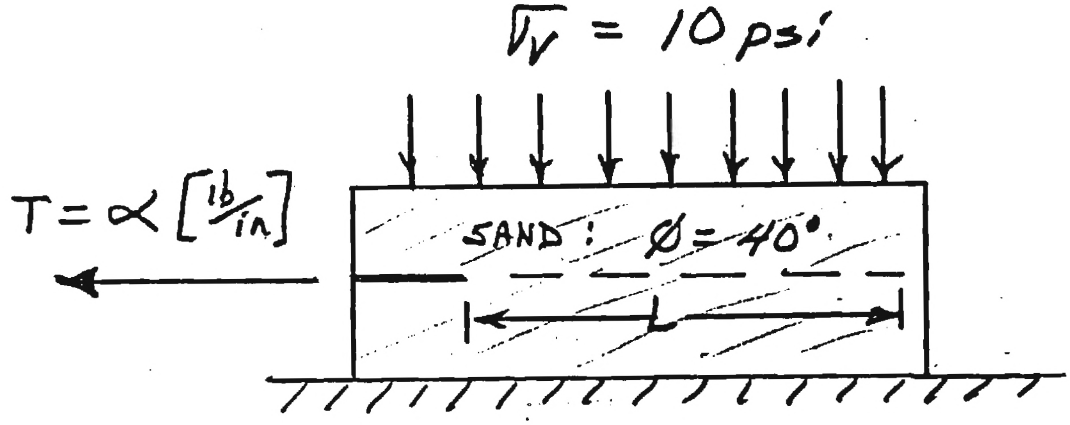
APPENDIX E.

MISCELANEOUS



280
260
240
220
200
180
160
140
120
100
80
60
40
20
0
APPLIED LOAD [lbs]
200
600
1000
1400
1800
2200
2600
3000
LOAD CELL RESPONSE [lbs]
CONFIRMS 10:1 RATIO
TENSION - CREEP FRAME
CELL CALIBRATION

Example to predict first occurence of rupture.



$$T = 2 \cdot L \cdot \sigma_r \cdot \tan\phi$$

$$L = \frac{T}{2\sigma_r \tan\phi}$$

$$L = \frac{542 \text{ lb/in}}{2(10 \text{ lb/in}^2)(0.84)}$$

$$\underline{\underline{L = 32.24 \text{ inches}}}$$

Wide Width tensile strength = 542 lb/in